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PRESTRESSED CONCRETE INSTITUTE

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Journal of the prestressed concrete institute

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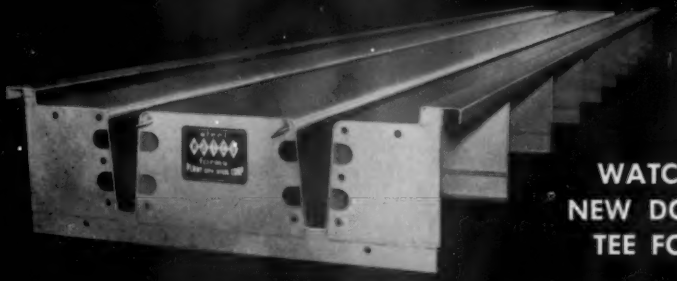

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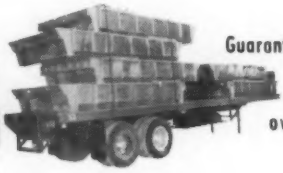

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Sept., 1961
Vol. 6, No. 3

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE.**

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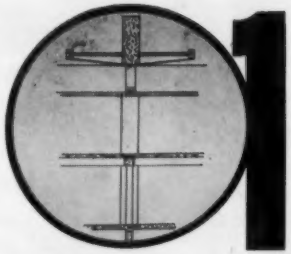

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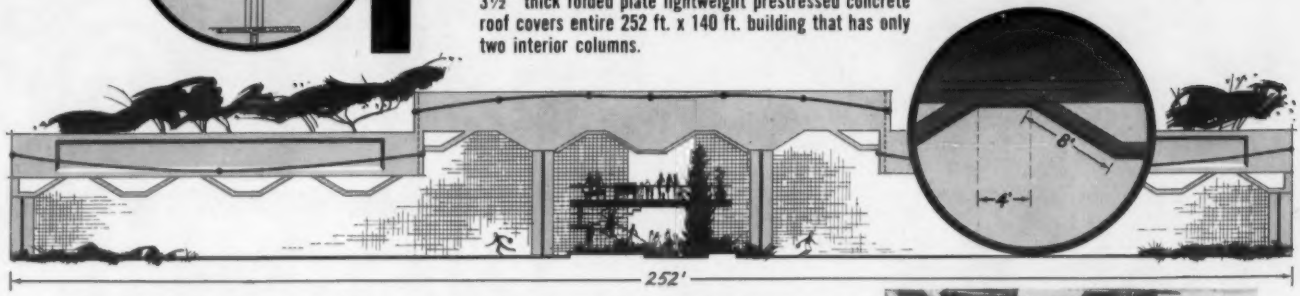

Three cast-in-place post-tensioned, prestressed concrete beams tied together with Prescon stressing tendons into one long unit make possible the unique structure of a 48 lane bowling alley that has no support columns in the lane area.

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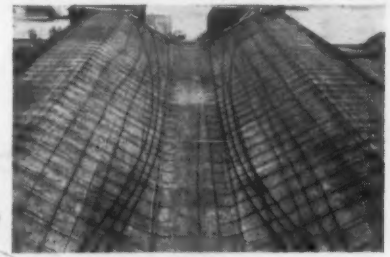

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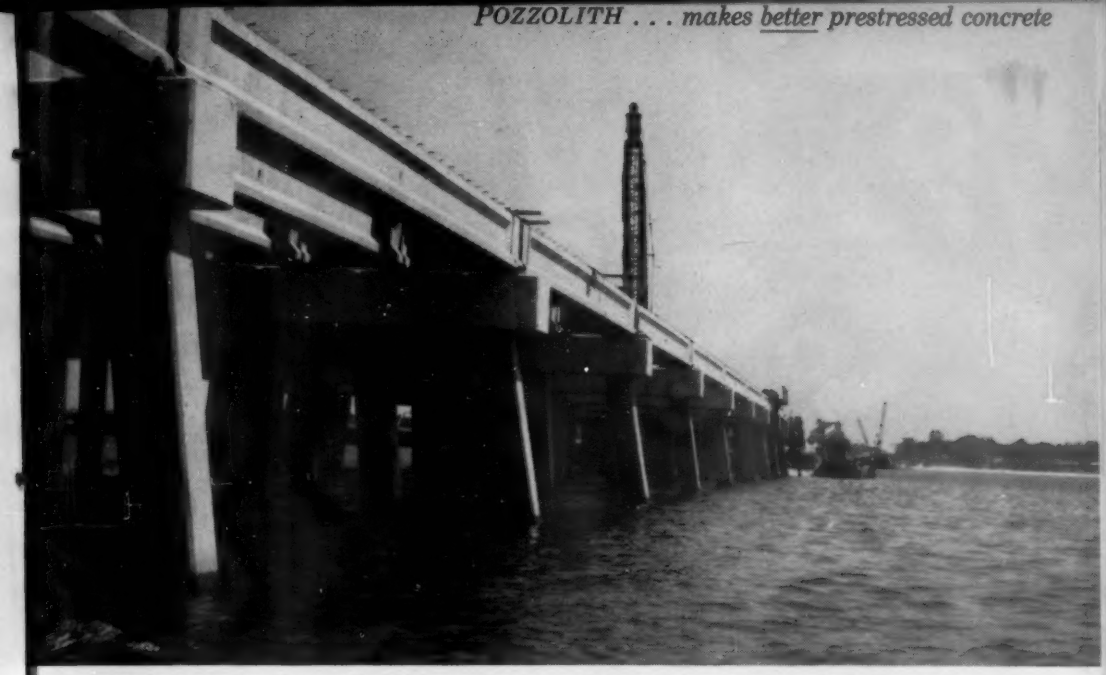

BILOXI BAY BRIDGE—67′ wide—will connect Biloxi and Ocean Springs, Mississippi with two 26′ clear roadways for 4-lane traffic. POZZOLITH concrete was used throughout the new prestressed structure—including piles, deck beams, deck slab, pile caps and footings. Construction Agency: Mississippi State Highway Dept. • General Contractor: J. B. Michael & Co., Inc., Memphis.

Big bridge over Biloxi Bay

new $6 million prestressed project spans 1 1/2 miles

When traffic starts to roll on the completed Biloxi Bay Bridge in early '62—over 39 miles of prestressed POZZOLITH concrete members will be in place. These consist of 128,000 linear feet of prestressed piles, ranging in length from 45′ to 95′ in four sizes—36″, 30″, 24″ hollow void and 20″ solid, plus 1,512 prestressed deck beams each 52 feet long. Total prestressed and poured-in-place POZZOLITH concrete for the project is 52,000 cubic yards.

A complete on-site prestress yard was set up by the contractor including a 7-lane casting bed and dry batch concrete plant. Three 330-foot lanes are for casting beams, four 465-foot lanes for piles.

Job demands and a tight timetable led to these requirements for the 5000 psi concrete specified: (1) Controlled retardation to assure workability and easy handling for the poured-in-place concrete (footings, pile caps, deck slab) after long transport by barge. (2) High early strength (3500 psi at 36 hours) with low W/C ratio for the prestressed members in order to maintain an extremely high production rate. (Example: 940 feet of piling per day.)

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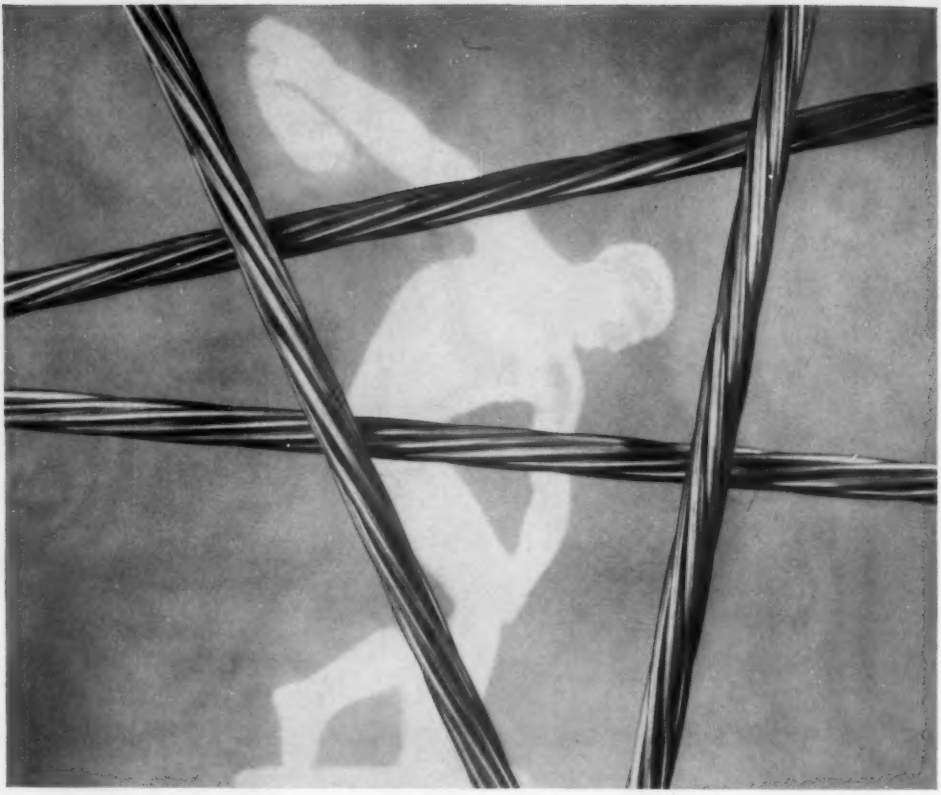

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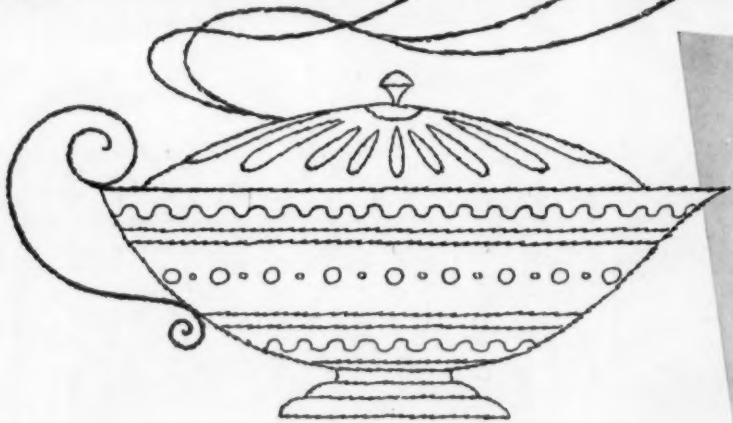

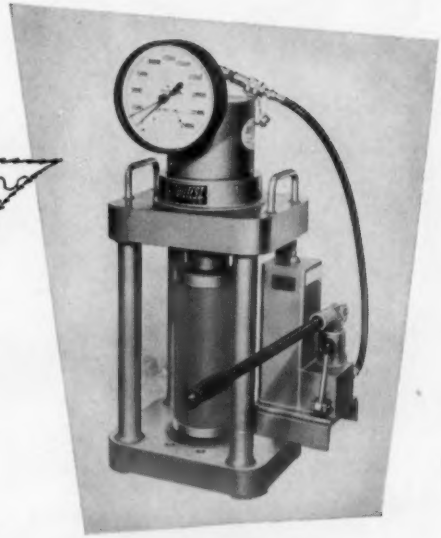

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Bethlehem Stress-Relieved Strand in 24,000 lineal feet of hollow prestressed-concrete piling for Seattle Freeway

High above Lake Union, in the Capitol Hill area of Seattle, Washington, a 2,700-ft section of the new Tacoma-Seattle-Everett Freeway is being rushed to completion. It consists of three pile-supported, raised roadways, side by side, each with 4 lanes. The center viaduct is reversible to handle peak traffic loads.

Production line system was set up by contractor. Several piles were made at one time, end-to-end. After curing cycle, they were separated by burning the strands between piles.

Morrison-Knudsen Co., Inc. and Rumsey Co., handling the contract as a joint venture, used over 900 tons of 7/16-in. diameter Bethlehem stress-relieved strand in the fabrication of 973 prestressed concrete girders and 405 hollow prestressed piles. A typical bent is shown above. The 5 cylindrical piles support 13 I-section girders, which are placed across the top of a cast-in-place cap. A concrete slab is then poured to complete the deck.

Leading prestressers are using Bethlehem stress-relieved strand because they like the results. They find that Bethlehem strand is flexible and easy to handle . . . and that they get the same dependable mechanical properties in every reel. Your nearest Bethlehem Sales office has full details.

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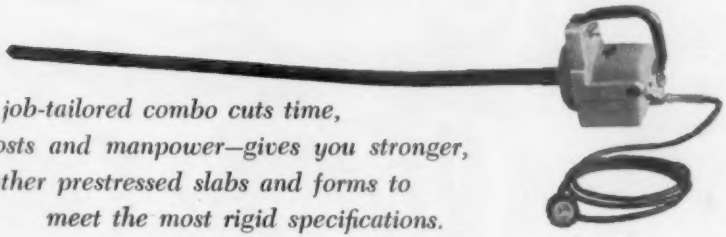

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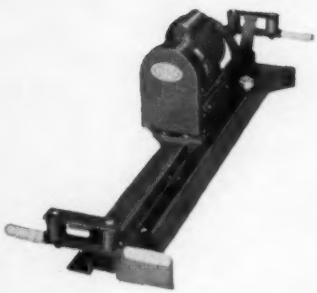

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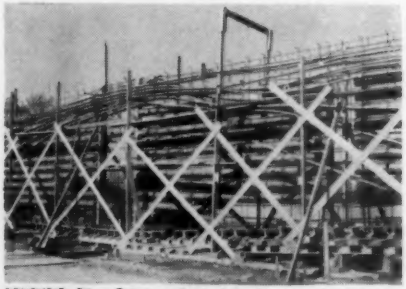
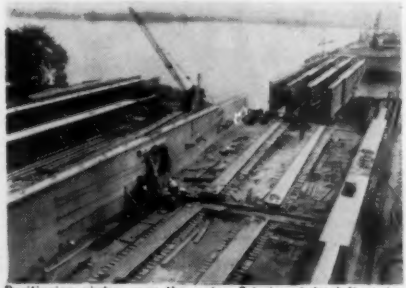


SONOVOID Fibre Tubes and steel tendons in place prior to placing forms. *Photo courtesy Terry Contracting Inc.*



Positioning girders over the water. Exterior girder left center. *Photo courtesy Sika Chemical Corp.*

The Oneida Lake Bridge near Brewerton, N. Y., is the longest prestressed concrete girder bridge in the U. S. There are two separate parallel structures, each carrying three lanes of traffic and having a pier-to-pier span of 320' 6".

Each span consists of twelve precast concrete cantilever girders, each 147 feet long and weighing 230 tons, which support five 231-foot, 230-ton concrete girders between them. To reduce dead load and simplify the moving and placing of the huge 230-ton exterior girders, they were voided with Sonoco SONOVOID Fibre Tubes, especially fabricated for this job.

SONOVOID Fibre Tubes are specifically designed to form voids in concrete floor, roof, and lift slabs, bridge decks, and in precast members. By displacing low-working concrete, they reduce weight and the amount of concrete and steel required . . . thus saving time, labor, materials, and money.

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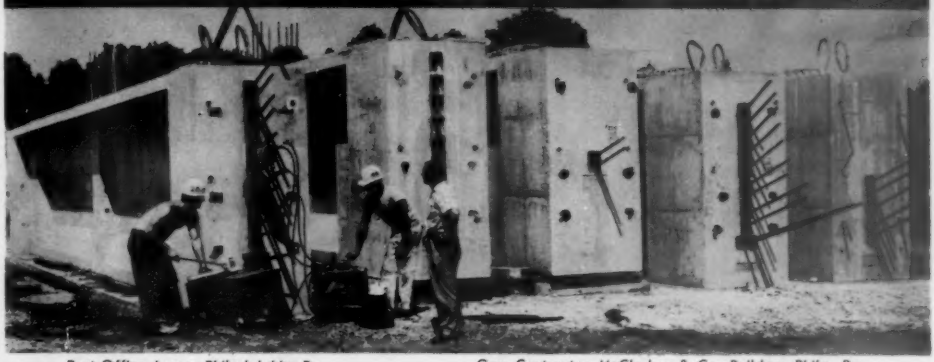

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THE PROBLEM: How can a prestressed concrete products manufacturer economically cast girders of unusual design, not readily adaptable to existing pretensioning beds?

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By casting the girders in another area and post-tensioning with STRESSTEEL BARS, Eastern Prestressed Concrete Corporation not only economically fulfilled the contract, but kept their pretensioning beds free for their regular production.

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GUEST EDITORIAL

Prestressed concrete, especially when precast, has opened up very great new possibilities for the architect—freedom by which members can be cast to take many different forms; and, more fundamentally, long-span construction with spectacularly small depth to span ratios which were, until now, not economically possible. Considered an experimental material in this country ten years ago, it has become almost completely accepted and commonplace today. If the material is to make further encroachments in the areas where it has not been used (and this will be done in the face of effective opposition by the structural steel industry) we should take a look in a constructively critical way at some of its areas to be improved.

CONNECTION OF PRECAST ELEMENTS: One of the greatest problems is the joining of pieces into an acceptably finished building. We have only to compare the elegance and precision of a rolled steel section welded to its columns with a similar detail in precast concrete to note how far we must progress to reach a similar elegance and precision. Much remains to be done so that the joints are compact and have a neat finished appearance. This requires more research, better engineering, attention to details, and more precise construction.

FINISH: It is possible, by proper formwork and control, to produce almost perfectly finished members, uniform in color and texture, yet many manufacturers do not do this. Once they realize that it can be obtained, architects will insist on perfect workmanship.

CONSTRUCTION: When prestressed concrete was in its developmental stage, many construction problems were tolerated which today are becoming inexcusable: excessive camber; failure of details in the stressing operation; poor performance by the suppliers of tendons, concrete and precast units. It is necessary that organizations engaged in prestressed concrete work organize and staff themselves in a way that is commensurate with the precise and difficult tasks they are facing.

JOURNAL in brief

Principles of Mechanized Production of Prestressed Concrete 14

Ben C. Gerwick

An excellent summary of present achievements in the field of mechanized production of prestressed concrete. Based on these observations the author developes the novel idea of a circular prestressed plant.

Theoretical and Practical Discussion of the Design, Testing and use of Pretensioned Prestressed Concrete Piling 22

G. C. Strobel and John Heald

With the updating of state specifications, driving and design procedures were developed which lead to the acceptance of prestressed piling. Load tests revealed surprising capacities for the prestressed piles and proved the bearing formulas very conservative.

Recent Developments in Automatic Manufacture of Prestressed Members in the USSR 34

V. V. Mikhailov

Faced with its huge backlog of new construction Russia has made great strides in developing fully automated and mechanized prestress plants. With machines such as the vibropuncher an automatic production line can produce up to 6,400,000 sq. ft. of floor panels annually.

Extrusion of Prestressed Concrete 47

Harry H. Edwards

The development of an economical machine produced prestressed section has been the aim of the industry for many years. The author traces the development of such a section and the current status of the industry.

Electrothermic Method of Pretensioning Bar Reinforcement of Precast Reinforced Concrete 57

B. G. Skramtaev

Of all the mechanized means of producing prestressed concrete in Russia none has had as much success as the electrothermic method. In great detail the author describes the method and its many applications.

Tall Precast Concrete Towers 72

Franz Sauter

Faced with a rigid time schedule, economic considerations, and limited equipment, prestressed concrete was selected for the 100 ft. light towers used for the World Amateur Baseball games in Costa Rica.

The Philosophical Creative Approach 75

Louis I. Kahn

The importance of emphasizing the structural frame in a building's architecture rather than suppressing it is this outstanding architects approach to creativity. He speaks of the joints in a structure as a strategic event and should be treated, symbolically, as if they were made of gold.

From Architect's Conception to Concrete Reality 80

Thomas J. Leidigh

The process of blending architectural expression, mechanical requirements, and a workable structural design and framing plan for the outstanding Pennsylvania Medical Center, are explained and illustrated by the author.

Concrete Results 86

J. B. Smythe

With the importance of final appearance and exactness of fit in mind for this 7 story precast structure, the casting of each individual member is carefully planned.

A Sliding Joint, Prestressed Concrete Pavement 92

Lyle E. Young

The writter presents a plan employing precast, triangular, diagonally prestressed slabs as another solution in the challenging area of prestressed pavements.

PROCEEDINGS PAPER

Principles of Mechanized Production of Prestressed Concrete

By
Ben C. Gerwick, Jr.*

Presented at the Prestressed Concrete Institute Convention
New York City, September, 1960

The ideal or ultimate system of mechanized production of prestressed concrete would be one in which the raw materials, aggregates, cement, water, mild steel, and prestressing steel entered the process near one end; while from the other end a completed, cured, prestressed concrete member would be loaded onto a truck. Also, ideally, within the actual production line it would be possible with relatively minor labor to change the length and the end details, so that while the resultant product was a standardized element, it could be varied within limits to fit many specific installations. At the end of the actual production process, the element would be stored in such a manner as to facilitate its loading and shipping.

This brief idealized statement points up one matter of special importance with prestressed concrete. That is that the approach to mechanization must include handling, storing, and loading as well as pouring, stressing, etc. The weight and size of individual elements are such that handling costs are an appreciable part of the overall costs.

No mechanization process can ignore or be separated from the concept of "flow of materials". The basic materials must flow to the process at the required rates and this material delivery must be mechanized to as high a degree as possible. Some automatic machines have been developed which have failed to provide adequate means of delivery of the concrete to the machine, a matter which is fully as important as that of placing and finishing the concrete.

One approach to mechanized production of prestressed concrete is an out-growth of experience with concrete block manufacture. In this approach, the product moves to the process. The concrete batching, mixing, and pouring is at a fixed location. Steel molds or frames move first to a stressing position where the steel is placed, stressed, and locked to the frame. The mold then moves on to the pouring position where the concrete is placed, vibrated, screeded, and finished. Then the mold moves on to accelerated curing, usually a steam tunnel. Emerging from this, the element is removed from the mold to storage while the mold is cleaned and returned to the cycle. The completed element may be palletized or otherwise combined within the storage area, and then moved to load-out.

When this approach is used with

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post-tensioning, the stressing and grouting of the steel are injected into the cycle before and during storage.

A typical example of such an approach is in the production of prestressed wall, roof and floor panels in factories in the U.S.S.R. (See Proceedings of World Conference on Prestressed Concrete, 1957, San Francisco, Page A-37-1, and Proceedings of the Third Congress of the Federation Internationale de la Precontrainte, Berlin 1958, Session III, Paper 1.) There are several types of automatic stressing equipment and methods in use in the U.S.S.R. One is a turning arm with a spindle, through which wire is paid out under tension. The wire wraps around pegs which are raised hydraulically in the proper sequence. Another machine, the DN-7, travels along the frame, and a spindle moves transversely to pay out wire under tension around fixed pegs.

Many of these factories are large, modern, well equipped plants, turning out a high annual volume of standardized products. Large steel frames, each the size of a complete panel, move through the many stages of the production process on rails or chain conveyors. The individual production steps have been mechanized. Each plant is vertically integrated, receiving aggregates, cement, mild steel, and high tensile wire, as its raw materials. Material flow around and within the plant is completely mechanized, although improvements are possible (and are being made) in plant layout.

More recently, electro-thermal stressing is being applied, with the strand being electrically heated while it is wound under moderate tension, with the remaining prestress added by the cooling of the wire. The use of polished hydraulic plungers to form voids and shapes is another development. Vibrators are combined with the plungers and presses.

Side forms of slabs are removed before the member enters the steam curing, the wire being anchored to pegs in the soffits that travels with the member during the production process. Push-button controlled automatic stacking is employed to feed the steam curing tunnel. This same basic approach has been applied to post-tensioned railroad ties in Germany. Here the bars are placed in a mold which moves along a conveyor to the pouring position, and thence to curing, stressing, grouting, palletizing and storage.

More recently, the American Concrete Crosstie Corporation announced development of an automatic tie machine, where the strand is positioned in the frame (mold) before pouring. The mold then moves on a conveyor to a pretensioning position, where the strand is automatically stressed. Then the mold is conveyed to the casting area, then on by conveyor to a curing tunnel, and finally to storage.

A leading American manufacturer of machinery has under development a machine which would be based on large multiple frames, which would act like moveable pretensioning beds and would travel through the several steps of manufacture. Thus each process, stretching strands, stressing strands, pouring, finishing, curing, etc., would always occur in a specified zone, thus permitting that process to be efficiently mechanized. Handling, storing, and load-out of the completed product are not inherently covered by this development and would require subsidiary means, as overhead bridge cranes, to carry these steps out in a mechanized or

partially mechanized manner.

In England, a variation of this approach is employed in the manufacture of prestressed railroad ties. Here wires are pretensioned on a number of parallel long-line beds. At a central point, concrete is poured into individual steel frame molds, which are then pushed lengthwise down the beds. In their final position, they are revibrated to insure bond.

A different basic approach to mechanization is for the process to move to the product. This is essentially the method used in the typical pretensioning plants in the United States today. Fixed beds are utilized. The strand is stretched and stressed in the bed. The concrete is batched and mixed in a central plant and brought to the point of pour. Pouring starts at one end of the bed and moves along it, followed by screeding and finishing. Steam-curing covers are placed, the product is cured, then the product is lifted out to storage and from storage to load-out.

Mechanized improvements already employed include the use of pouring machines that feed, vibrate and screed the concrete. Hydraulically actuated forms have been used. The stretching of the strand may be semi-automatic. Forms are designed that require little or no stripping. Automatic brush cleaning of the forms is employed.

Extrusion or slip-forming is gaining in popularity. The entire sides may be extruded or only the top portion. An interior void may also be slip-formed, using a long polished steel tube. For best slip-forming work, the concrete must be placed and compacted mechanically. A machine has been developed which combines strand positioning, places the mesh, mechanically places concrete, vibrates, compacts, and finishes it, while slip-forming sides and voids.

A mechanical concreting system from Germany, introduced into this country in Wisconsin over five years ago and recently installed in a modern plant in New Jersey, represents a high degree of mechanization. Fixed abutments are used to anchor and stress the longitudinal strand. A gantry crane straddles several beds and as it travels, it completes the product, one line or bed at a time.

The machine supported from the gantry positions the strands, slip-forms the exterior sides, forms the inner cores, pours the concrete in three layers, screeds and finishes the surface. Continuous cored planks are cast on top of each other, one layer on each bed per day. After approximately five days run, and a two-day wait for the top member to cure, the stress is released and the planks are cut to length with a saw. They are then lifted sidewise to a trailer truck and hauled to storage. (See Engineering News Record, May 26, 1960, page 34)

Such a major installation points up the possibilities of mechanization in two directions. First, it is substantially a complete mechanization of the forming and concreting phases of prestressed concrete production. While many different approaches and systems will be developed, and applied to different shapes, it is reasonable to conclude that none will show substantial improvement in this forming and concreting phase.

On the other hand, this type of installation also shows, by what it has not mechanized, the tremendous opportunities for further development. This is particularly true of the break-out, handling, transporting to storage, storage and load-out opera-

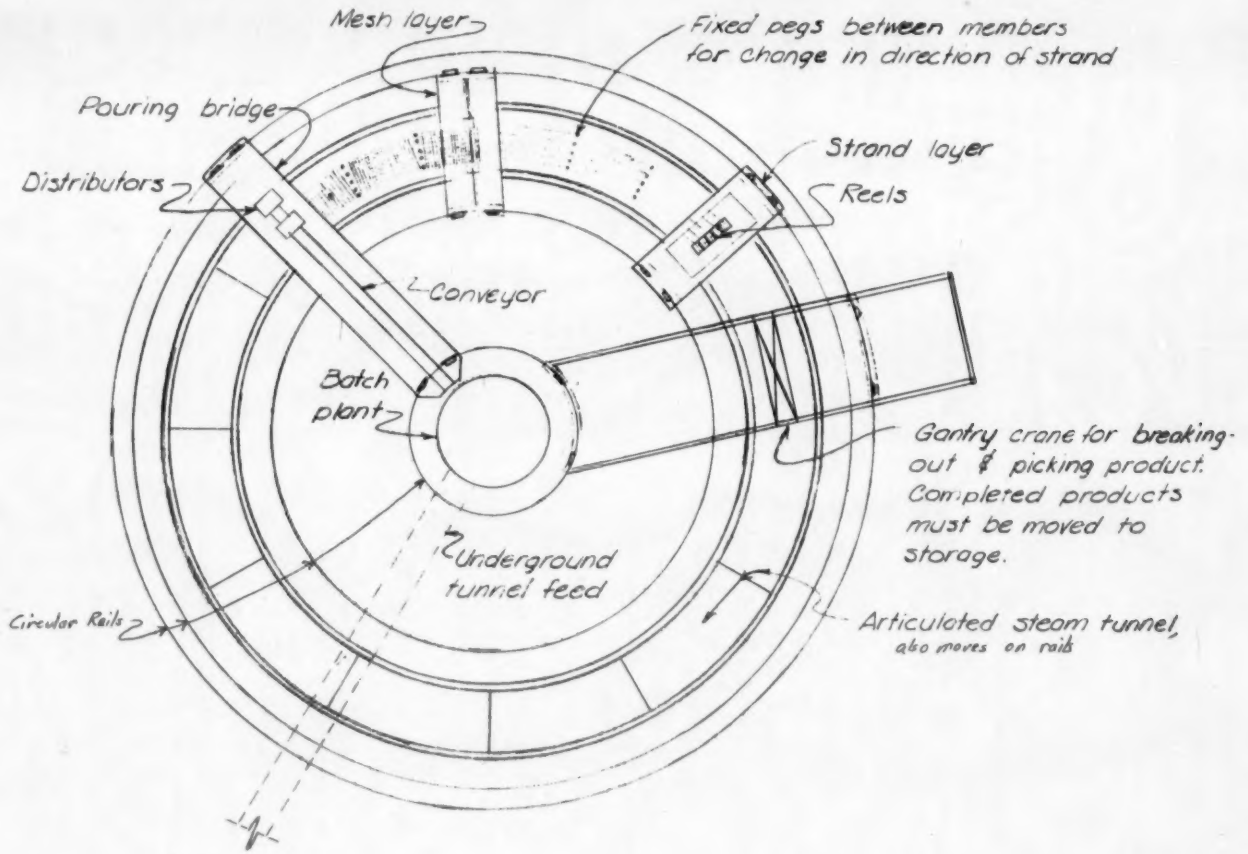
Mesh layer
Fixed pegs between members for change in direction of strand
Pouring bridge
Distributors
Strand layer
Reels
Conveyor
Batch plant
Gantry crane for breaking-out & picking product. Completed products must be moved to storage.
Underground tunnel feed
Circular Rails
Articulated steam tunnel, also moves on rails

Fig. 1

tions. To a lesser degree it is true of the strand stretching, anchoring, and stressing operations, which are only partially mechanized. But this installation undoubtedly represents a significant step forward in prestressed concrete manufacture and is indicative of a pattern which appears imminent in our industry.

A further development following this same basic approach (process moving to product) is shown diagrammatically in fig. 1, a circular plant layout.

Assuming a rate of production of one foot per minute, and a 16 hour cycle, the production track must have a diameter of 300 feet. The storage area becomes a great circular area all around the plant. This almost dictates a carrier, fork lift, or similar mobile machine for handling. When the product has been lifted out by the track-mounted gantry, it may be moved to storage by conventional bridge cranes, carriers, fork lifts, or travel lifts.

While the majority of pretensioning beds in the United States are long-line (300 ft. to 600 ft.); increasing use is being made of pretensioning under a gantry. The beds may then be short-line, as described in my paper presented at the Berlin Congress of the F.I.P. in 1958, or may be long-line, with the lines parallel to the gantry tracks. In the latter case, two bridge cranes are employed with both lateral and longitudinal movement, so as to permit storage under the gantry.

Examining the short-line process first, this has the obvious problem of requiring more anchoring and stressing per element produced than in a long-line. But it greatly facilitates handling, storage and load-out. Also, using the deflected strands technique, the problems of exact tensioning are greatly reduced on such a short-line bed, and can be more accurately performed.

If we are able to work on a 24-hour cycle, then in such a "short-line" plant, we can have an end work area where strand is pre-cut to exact length, and here mild steel is preassembled into cages. The cycle is then as follows: Light gantry crane places cages and strand. Strand is pretensioned. Concrete comes in from side and is transported laterally and poured by a machine. In the ideal plant, the pouring machine would have hopper capacity for one transverse run. This machine would screed and finish in one operation, or a second machine would follow for finishing and screeding. The light gantry crane would also place and remove steam covers. The heavy gantry crane would break out units to storage and load-out for transportation. Load-out will almost always have to be a day-shift operation. So break-out may be scheduled for a night-shift.

If we consider a basic single-shift operation, then one gantry can do the job. The gantry itself will work practically around the clock, setting cages and loading-out on the day-shift, setting steam hoods on the second shift, breaking-out on the third shift. This either reduces curing time to a total of about 16 hours including holding-time, or we need a two day supply of beds. In the latter case, it may be desirable to have a central rather than an end cage assembly area, and work one day east, the next day west, and so on.

As a detail alternative, the strand may be fed to the beds from a swift car operating on tracks parallel to and outside the gantry rail.

An overhead bridge crane is superior to a gantry in that the abut-

ments of the bed can be outside of the gantry rail, thus increasing the length of element which the crane can handle. But such a bridge crane requires two pick-up travelers so as to pick both ends of a member. Of course, a strong-back can also be used.

On a three-shift operation, and a 16 hour total curing time, the heavy gantry must operate on separate outside tracks, while the light gantry passes underneath, running on an inside pair of tracks. See Fig. 2.

Assuming an end cage assembly area, then the cage handling and stressing might proceed westward from this area for a 24-hour period, then return. The concrete pouring would likewise start, say 3 hours later, proceed westward to the end, then return to the east end to start another cycle. Steam hoods could also be handled by the light gantry, picking from the 16-hour point (to start), moving to the east end.

The heavy gantry would start breaking out cured elements at the 16-hour point, move these westward to storage. When it reached the 24-hour point (west end) it would be breaking out from the east end again.

This involves handling the completed elements over new work (and men) which does not contribute to efficiency. A central cage assembly area plus storage and load-out at both ends will minimize this problem.

Depressing the beds below the gantry rails is one possibility and will serve several purposes:

1. Enable abutments to be outside the rails.
2. Facilitate dumping of concrete to hopper.
3. Possibly facilitate steam curing, protection, etc.

Suitable provision must be provided for cleaning and drainage, and the depressed area must be large enough to facilitate access.

Longitudinal (long-line) prestressing under gantries or bridge cranes is also practiced. It is capable of some mechanization, by adaptation of the above methods. Concrete would come in from the sides at intervals by truck. A pouring machine would travel transversely. Strand would be fed out from one end, stretched, and stressed.

When cured, units would be handled sidewise, to side storage under the gantries, or lengthwise. Under this scheme, two gantries must cooperate on most lifts, unless the members produced are relatively short. A variation of this scheme that has been employed is to pick

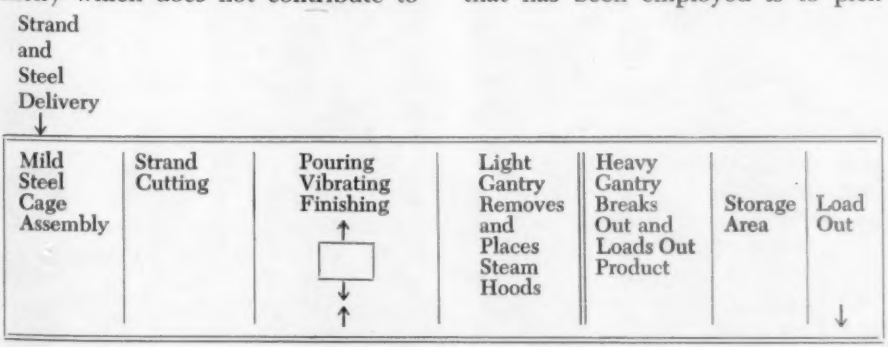

Figure 2

all members from the beds and move them sidewise to a flat car which transports them longitudinally to the storage and load-out area.

One other basic approach to mechanization is for the product to move through the process. In this approach, the completed product is pulled through, thus stressing the strands which are anchored in the completed product. The other end of the strands, a full cycle away (perhaps 1500 feet), are fed off reels, which are braked to provide the necessary tension. Concrete is fed in from the side and poured and vibrated at one point. The forms either travel with the freshly poured element until initial setting, or the sides are extruded and the soffit moves. This soffit continues on through the curing period. The soffit and side forms may be of continuous belting on rollers.

After emerging from the curing process, the units are cut to length, (if not divided in manufacture), lifted sidewise by a bridge or gantry crane to storage and load-out.

The application of a constant high stress to the completed product is by either side friction or mechanical gripping devices, the pull being applied through continuous jacking or direct geared drive.

The braking of the reels may be directly connected to the driving gears or dynamic (electric) braking may be used with the power fed to the driving motors. Thus the only new power needed, other than losses, is to overcome friction of the product in the line. By arranging the entire line on a slope, gravity can be employed to provide the basic force, augmented and controlled by a powered drive. (Patent applications have been filed for the above described process.)

There remains another problem for which a satisfactory solution is needed. It is the forming of voids. Progress has been made with polished steel mandrels, paper boxes, fiber tubes, paper wrapped stressed elements, pneumatic mandrels, and mechanical mandrels. But none appear to be one-hundred percent satisfactory at present; problems include the introduction of undesirable tensile stresses in the fresh concrete, and plastic flow. The present efforts along all the above lines should eventually result in a satisfactory solution that can be entered into a mechanized production process.

The development of mechanization can only go hand-in-hand with the development of suitable standardized products and the marketing (sale) of sufficient quantities to utilize a mechanized plant's output and to justify the capital expenditures required.

Products must be simplified and every effort expended to reduce obstructions to mechanization, as for example, the elimination of end-blocks. This does not mean that we cannot mechanize a unit that has end-blocks, but merely that such changes in section are a further difficulty to overcome.

The ideal product for mechanization in any industry is one which can be utilized in many different structures and uses, that can be altered (in length, for example) with little difficulty and that can be combined or connected readily.

A decade ago, it was thought that this requirement would be met by prestressing concrete blocks together in different arrangements and lengths, but this proved to be uneconomical. The basic unit, the block, represented too small a part of the entire process. One such system that has survived and grown

has been the Raymond system for manufacture of the concrete cylinder pile. Large concrete pipe units, generally 16 feet long, are connected end-to-end by prestressing to form piles of varying lengths. In this case, the individual unit is sufficiently large to represent a significant part of the entire production process.

Similar efforts in this direction have been made in New Zealand, Australia, and the U.S.S.R. where standard bridge girder sections have been cast in 6 foot or longer lengths, to be subsequently connected by prestressing. But in these cases, the mechanization has been primarily confined to the manufacture of the precast units, with the prestressing essentially a field operation.

Of course, many buildings and other structures have been constructed from precast units, connected in the field by prestressing. However, most of these have been individually designed units, and manufacture has been mechanized to only a minor degree.

This concept needs further intensive development. Relatively large individual units appear desirable; perhaps these will even be pretensioned. Therefore, the basic schemes of mechanization discussed earlier will apply to the manufacture of the individual units.

There is also a great deal that has and can be done to make multiple and varied use of standardized prestressed concrete members. The examples of the double-tee and single-tee in this country show how a single standard product can be sufficiently widely utilized to justify mechanized manufacture.

Because of the urgent need for better production processes, there has been a tendency to concentrate on improving existing schemes only, and to mechanize detailed tasks, rather than give consideration to entirely new approaches. This is the purpose for this paper, the hope that it will serve as a guide and stimulant to fresh approaches.

Theoretical and Practical Discussion of the Design, Testing and Use of Pretensioned Prestressed Concrete Piling

by G. C. Strobel[1] and John Heald[2]

INTRODUCTION

Many foundations for highway bridge structures in Nebraska and other Great Plains States in the United States rest on piling. Most frequently, rock formations are at such great depths that it becomes necessary to use friction or displacement piling which obtain their support strength from strata and soils through which the piling are driven.

The search for a satisfactory economical displacement pile in Nebraska has been a constant and continuing process. Piling were either incapable of large design or carrying capacities, uneconomical, or too heavy and massive for light weight pile driving equipment. The average small bridge projects, for which normal pile driving equipment must be designed, are projects on which from 2,000 to 4,000 lineal feet of piling per bridge are required.

In 1954, three factors contributed greatly to the initial use of prestressed concrete piling on highway bridge structures in Nebraska. These factors were, the unsatisfactory driving of various types of displacement piling; the introduction and use of prestressed concrete piling in the United States; and the major revision and updating at that time of construction methods and standard specifications for the Nebraska Department of Roads.

The problem of unsatisfactory driving of various types of displacement piling was serious and costly as borne out by frequent pile collapses in driving and numerous broken, split and twisted piling. These failures had to be extracted. Precast concrete piling had been used for some time, however their great mass contributed to the early acceptance of the lighter and more easily handled prestressed concrete piling.

As mentioned the third factor which contributed greatly to early acceptance and use of prestress concrete piling was the revision of standard specifications in Nebraska. This actually turned out to be a key factor in the satisfactory use of the product. The old 1937 standard specifications, to which many subsequent revisions had been made, still permitted the use of swinging pile driving leads for what was considered the normal or small pile driving project. This was the source of many pile driving difficulties. The old standard specifications also permitted the use of gravity type hammers. The revised specifications required that pile driver leads be constructed in such a manner as to afford freedom of movement of the hammer and shall be rigged to insure support of the pile during driving. The support of the pile during driving was to be obtained by

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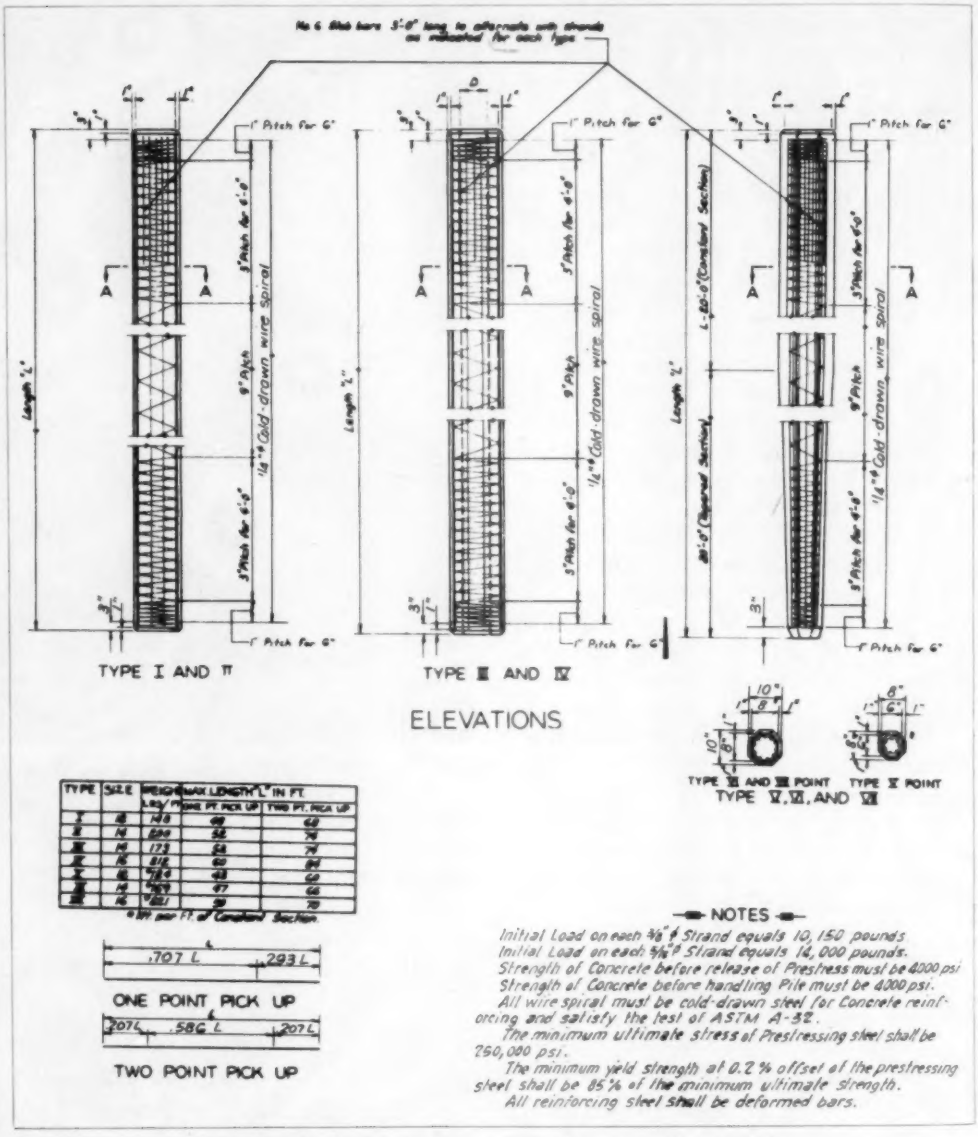
the requirement that the leads not be hung from lines, but that the bottom of the leads should be supported on a firm foundation. Additional requirements of the standard specifications required the use of steam, air, or diesel type pile driving equipment for concrete piling and a driving capacity depending upon the mass to be driven. These specification requirements contributed to early success in the use of prestressed concrete piling because they also formed the basic criteria essential to success in driving these piles. These specifications required contractors throughout the state to convert to equipment large enough and, which actually had a capability of driving concrete piling. Since very little steam and air pile driving equipment was being used in the state at that time, most contractors purchased compact operating diesel pile driving units. This also appeared to be a major factor in early successful use of prestressed concrete piling because the diesel drivers had the ability to drive the massive piling through difficult driving strata without any apparent damage to the pile.

Fig. 1. Driving 90 ft. long 14 in. octagonal prestressed pile with a M-K diesel hammer.

DESIGN

In order that a satisfactory handling and driving pile would result, certain basic preliminary design assumptions were necessary. No previous criteria were available, but after numerous preliminary calculations, the basic criteria was adopted to use a design concrete prestress force of 600 to 700 pounds per square inch in the piling. This insured sufficient force to properly handle and drive the piling. The accuracy of this decision is borne out by the successful handling and driving of hundreds of piling, and the acceptance of this criteria by other states who have begun the use of prestressed piling. A reduction in the initial prestress force would have increased the handling problems although, on piling driven to rock, more load carrying capacity would be available.

Figures 2 and 3 indicate an acceptable prestressed concrete pile standard plan on which twelve-inch square to sixteen-inch square, solid and hollow piling are shown, plus octagonal types. The square solid types have been used successfully in plastic and point bearing stratas. The octagonal tapered piling are used in sand, sandy silts, and sand-gravel formations. The hollow square piling have been used with limited success. Due to the critical placement requirements of the void during the manufacture of hollow piling,


TYPE I AND II
TYPE III AND IV
ELEVATIONS
TYPE V AND VI POINT
TYPE V POINT
TYPE V, VI, AND VII
.707 L
.293 L
ONE POINT PICK UP
.207 L
.586 L
.207 L
TWO POINT PICK UP
NOTES
Initial Load on each 3/8" ϕ Strand equals 10,150 pounds.
Initial Load on each 1/2" ϕ Strand equals 14,000 pounds.
Strength of Concrete before release of Prestress must be 4000 psi
Strength of Concrete before handling Pile must be 4000 psi.
All wire spiral must be cold-drawn steel for Concrete reinforcing and satisfy the test of ASTM A-82.
The minimum ultimate stress of Prestressing steel shall be 250,000 psi.
The minimum yield strength at 0.2 % offset of the prestressing steel shall be 85 % of the minimum ultimate strength.
All reinforcing steel shall be deformed bars.

TYPE	SIZE	WEIGHT/FT	MAX LENGTH "L" IN FT. ONE PT. PICK UP	TWO PT. PICK UP
I	8	140	99	68
II	10	220	92	74
III	10	173	84	74
IV	12	215	90	84
V	12	270	83	80
VI	14	329	77	66
VII	14	381	79	70

* Wt per Ft. of Completed Section

Fig. 2. Standard plan of precast prestressed concrete bearing piling.

minor flotation has been encountered. This produces an eccentric pile which causes eccentricity in driving and minor tension cracks can occur transverse to the axis of the piling.

Since competitive piling were permitted the use of pile splices to reduce the length of driver leads required in the pile driving, it appeared reasonable, that prestressed concrete piling should also be permitted an approved splice. For this reason sleeve pile splices were developed and tested in flexure by testing a 10 ft. long simply supported section. Deflections were measured at three points along the span.

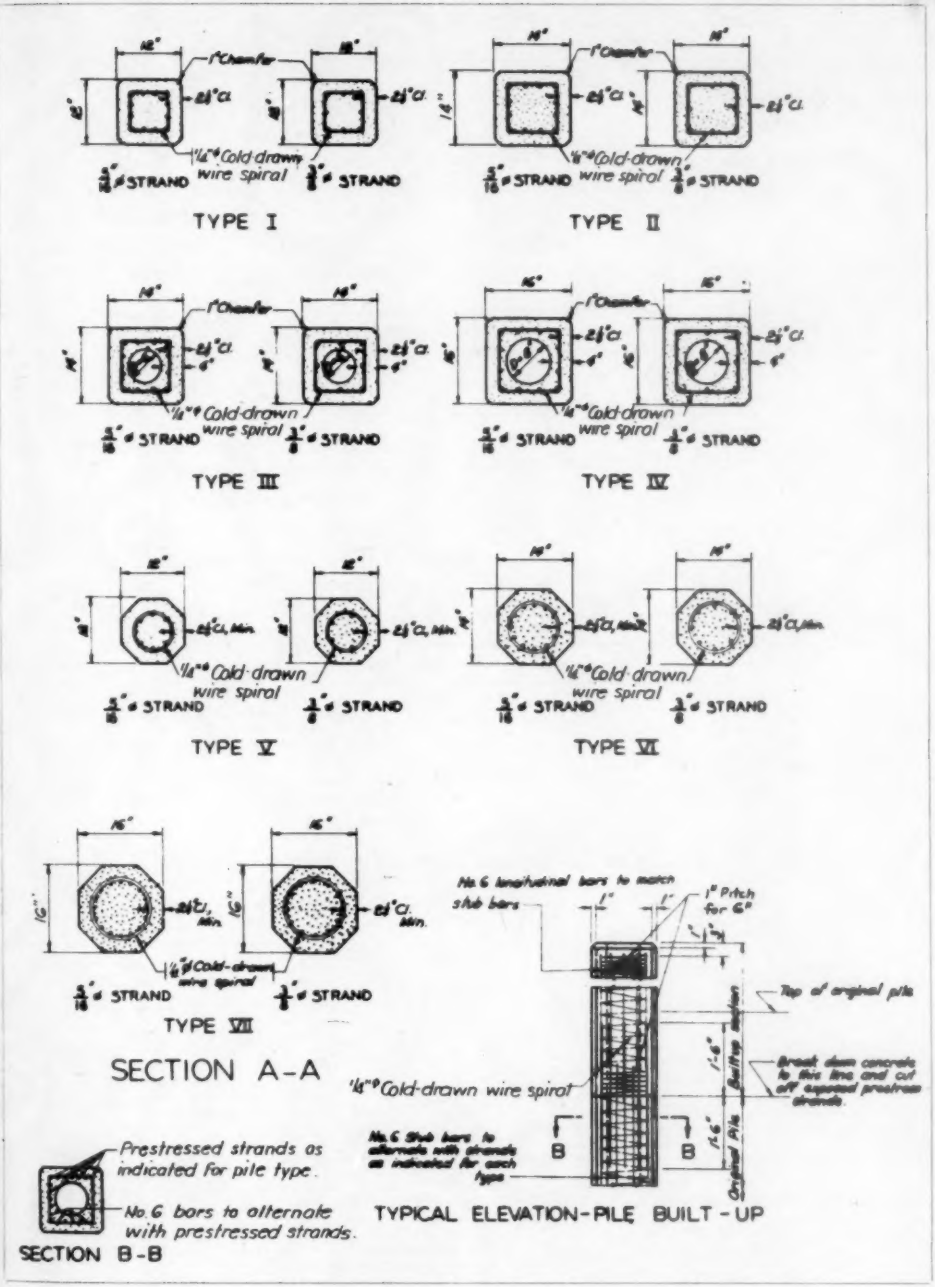


Fig. 3. Standard plan of precast prestressed concrete bearing piling.

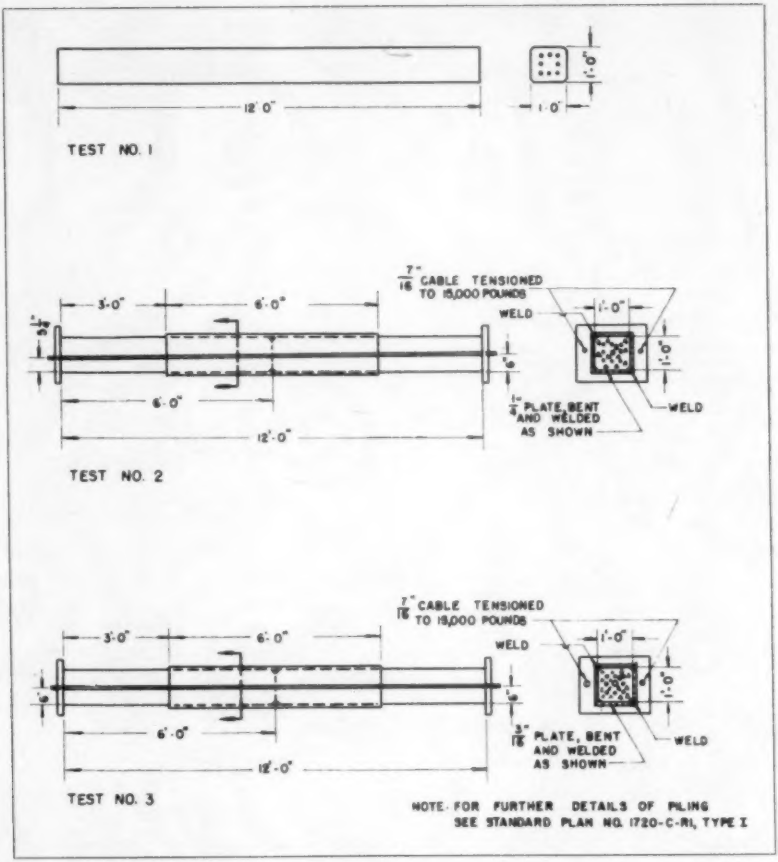


Fig. 4. Test specimens used for flexure tests of prestressed concrete piling.

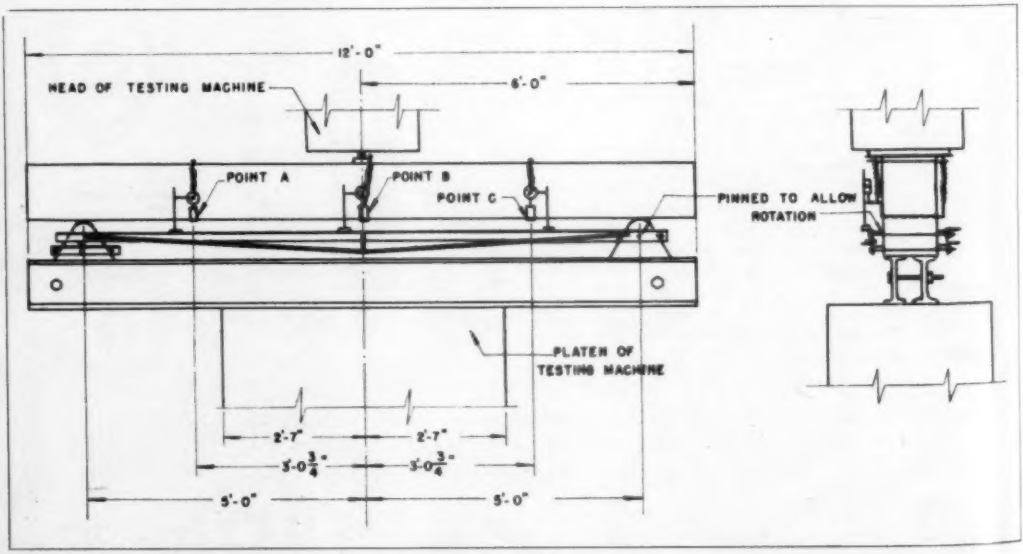


Fig. 5. Details of testing apparatus for flexure tests of prestressed piling.

Tests were made of two-6 ft. pile sections joined by a 6 ft. sleeve, which was fabricated from ¼ in. thick steel plates for one test and 3/16 in. thick steel plates for a second test. Figure No. 4 shows the type of sleeve splice which was tested and indicates the use of 2 7/16 in. cables. These cables were tensioned to 15,000 pounds each, which simulated an end bearing of 15 tons per pile. This appeared to be the minimum which might ever be specified in future bridge projects. Figure No. 5 indicates the test setup and positioning of the deflection gages and Figure No. 6 is a graph of Load plotted against Midspan Deflection for the two splice sleeves being tested and for the test of a 12-inch square pile without a splice. These tests indicated a desirability to specify a ¼ in. thick steel plate sleeve or perhaps one fabricated from even heavier plates, if an unspliced condition is to be assimulated.

PILE LOAD TESTS

The simultaneous introduction of prestressed concrete piling and diesel pile drivers caused a number of problems relative to driving correlation. The actual energy delivered by diesel hammers to a pile is variable and frequently slightly in doubt.

In order to gain some information to the solution of this problem, the Nebraska Department of Roads conducted numerous pile load tests to obtain some insight into the energy being delivered to the pile under different conditions of driving. Figure No. 7 shows typical Load-Settlement curves developed to arrive at usable pile driving formulas and to establish reasonable energy figures for diesel hammers.

The pile load test shown in Figure No. 7 indicates that the calculated bearing by formula should be,

$$P = \frac{4.6\,E}{S+0.1} \times \frac{(W)}{(W+M)}$$

$E = KWH$

$$K = \frac{1.8\,(H-1)}{(H_m)}$$

H = stroke in feet

H_m = 8.0 feet

P = safe load in tons

W = weight of hammer

M = weight of pile plus weight of cap in tons

E = energy per blow in foot-tons, and S = average penetration in inches per blow for last 10 blows.

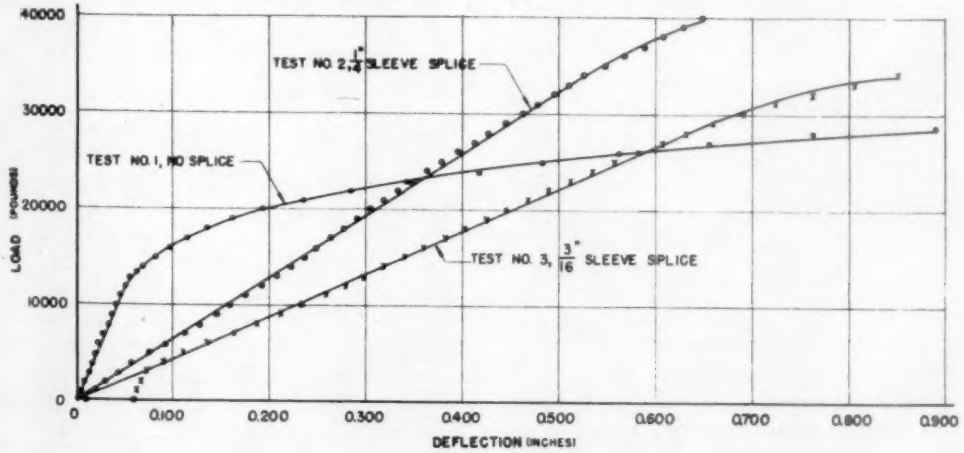


Fig. 6. Midspan deflections during flexure tests of prestressed piling.

Subsequent load tests and the need to simplify engineering inspection in the field has resulted in adaptation of the following pile driving formula for steel "H", steel shell and timber piling:

$$P = \frac{3.0\,E}{S+0.1} \times \frac{W}{W+M} \qquad (2)$$

Concrete piling bearing calculations have been simplified to,

$$P = \frac{7.0\,E}{S+0.1} \times \frac{W}{W+M} \qquad (3)$$

The energy (E) denoted in formulas (2) and (3) above for certain diesel hammers is as follows:

Type of Diesel Hammer	Energy
Link-Belt 12K	7.5 foot-tons
McKernan-Terry DE-30	See table for stroke
Delmag D-12	See table for stroke

Energy in foot-tons for McKernan-Terry DE-30 or Delmag D-12 hammers vary in accordance with the following table:

H (Stroke)	E (Energy)
8 Feet	10 Foot-tons
7 Feet	9 Foot-tons
6 Feet	8 Foot-tons
5 Feet	7 Foot-tons
4 Feet	6 Foot-tons

In addition to load testing of piling to substantiate design or calculated bearings, at some major projects, where quantities are sufficient to justify the expense, piling are tested to determine design pile bearings for plan preparation prior to taking of bids to avail the State of savings realized by keen competition. One such load test program was conducted at 72nd and Grover Streets in West Omaha, Nebraska. The following tabulation gives some results determined from the pile driving data:

TABULATION NO. 1

No.	Kind of Pile	Penetration below ground	Yield[a] (tons)	Calculated Bearing (tons) by Formula	Calculated Shear[b] (tons)	Design Bearing[c] (tons)
1.	Step Taper, Concrete Filled, 14⅜" Butt, 8⅝" Tip	45'-4"	90	52[e]	112.3	35
2.	Step Taper, Concrete Filled, 16⅜" Butt, 10⅜" Tip	43'-1"	105	60[e]	124.2	35
3.	Monotube, Concrete Filled, 12" Butt, 30' Tapered Section	79'-4"	220	80[f]	205.8[d]	70
4.	14" Octagonal Prestressed Concrete 20' Tapered to 10" Tip	43'-0"	195	48[e]	128.1	70
5.	12" Diameter Straight Sided Shell, Concrete Filled	43'-0"	85	35[f]	113.7	35
6.	Monotube, Concrete Filled, 12" Butt, 30' Tapered Section	43'-1⅞"	90	43[f]	97.8	35

7.	10″ x 42 lb. Steel "H"	43′-0″	55	35[f]	114.4	[g]
8.	10″ x 42 lb. Steel "H"	79′-10½″	165	34[f]	224.4[d]	55
9.	12″ Diameter Straight Sided Shell, Concrete Filled	79′-9″	160	90[f]	238.2[d]	55
10.	14″ Octagonal Prestressed Concrete 20′ Tapered to 10″ Tip	79′-11½″	305	40[e]	265.5[d]	70
11.	12″ Square Prestressed Concrete Uniform Section	43′-2″	95	33[e]	144.8	35
12.	12″ Diameter Drilled-In-Place Concrete	43′-0″	95			35

NOTES:

a. Yield is based on maximum load supported by pile with net settlement of one-forth inch or less.

b. Average undisturbed shear strength (s) is equal to one-half the unconfined compressive strength.
Point resistance = 9 sA (silts)
= 18 sA (sands)

c. Design bearings used in the actual footing pad designs detailed on the construction plans.

d. s = 0.80 T./sq. ft. From 0-77 feet
s = 1.90 T./sq. ft. From 77-80 feet

e. $P = \frac{4.5\ E}{S+0.1} \times \frac{W}{W+M}$
P = safe load in tons,
W = weight of hammer or ram in tons,
M = weight of pile plus weight of cap in tons,
E = energy per blow in foot-tons,
and S = average penetration in inches per blow for last 10 blows.

f. $P = \frac{3\ E}{S+0.1} \times \frac{W}{W+M}$
(Notations as above)

g. This pile was not incorporated in the design.

Figure No. 8 shows plottings of the preliminary drive sample data, pile driving information, and load-settlement data for one pile in the test pile group in West Omaha. Similar data was developed for all piling in this program.

It is only through the continued programs of additional and advanced research, such as pile load tests to substantiate calculated results and competitive pile testing in similar stratas that more useful information will be gained for substructure designs. This data and accompanying economic comparisons will make better future use of pile capacities.

MANUFACTURING

The majority of prestressed concrete piles are manufactured by the long line method, using multiple strand jacking for the prestressing

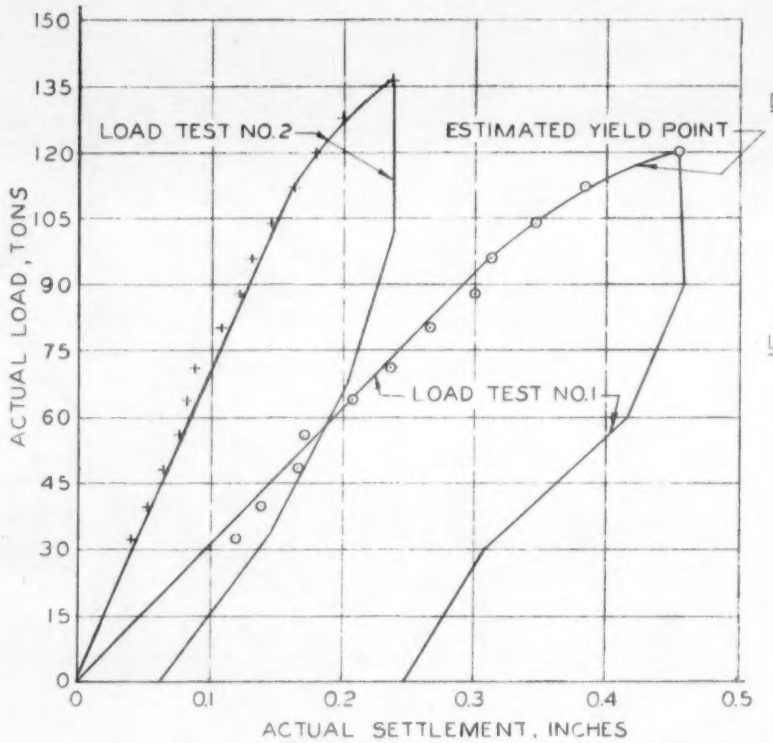
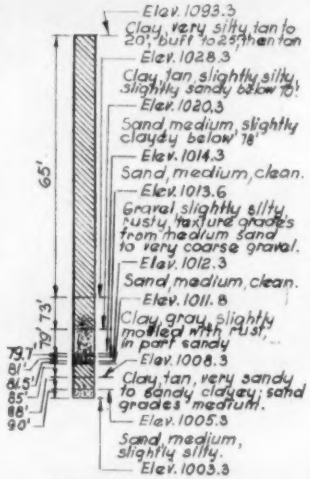
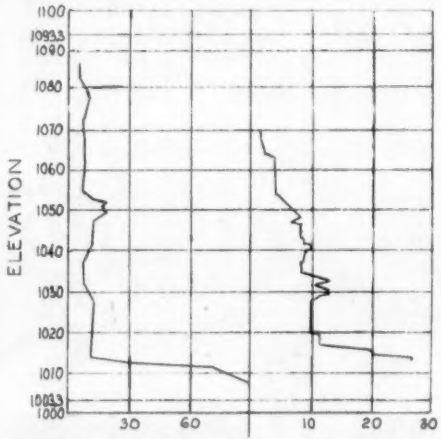
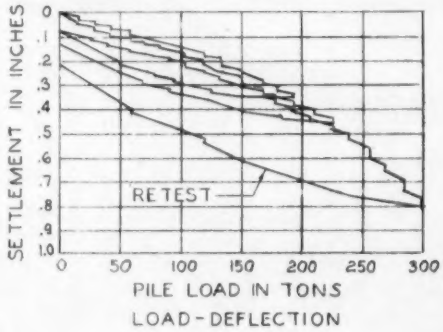

NOTES: —o— LOAD TEST NO.1 PILE SHELL EMPTY
—•— LOAD TEST NO.2 PILE SHELL FILLED WITH CONCRETE
PILE LENGTH = 49 FEET.
PILE SHELL DIAMETER = 12 ¾"
PILE SHELL THICKNESS = 0.14"
LOADING DATA
L.D. TEST NO.1 INITIAL LOAD WAS 32 TONS THEN INCREASED BY 8 TON INCREMENTS AT 1 HR. INTERVALS TILL 120 TONS WAS REACHED WITH PILE SHELL EMPTY. 120 T LOAD HELD FOR 26 HR. 45 MIN. SETTLEMENT FOR LAST 24 HR. WAS 0.11" NO SETTLEMENT FOR LAST 17½ HR.
L.T. TEST NO.2 INITIAL LOAD WAS 32 TONS THEN INCREASED BY 8 TON INCREMENTS AT 1 HR. INTERVALS TILL 136 TONS WAS REACHED WITH PILE SHELL FILLED WITH CONCRETE. 136 T LOAD HELD FOR 24 HR. SETTLEMENT WAS 0.032" IN THAT 24 HRS.
LOAD SETTLEMENT CURVES
PILE NO. 3 PIER NO. 2
PROJ. SG-187(6) DOUGLAS COUNTY
STATE ROAD BENNINGTON - OMAHA
DATE JULY 1957
PLOTTED BY C.S.
LOAD TEST NO.2
ESTIMATED YIELD POINT
LOAD TEST NO.1
ACTUAL LOAD, TONS
ACTUAL SETTLEMENT, INCHES
0
15
30
45
60
75
90
105
120
135
150
0.1
0.2
0.3
0.4
0.5

Fig. 7. Load—settlement curves for determination of pile driving formulas.

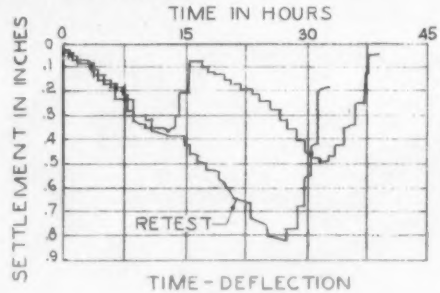


HOLE NO. 2
15' South of South Test Pile





LOAD - DEFLECTION



TIME - DEFLECTION

PILE DATA

14" Octagonal Prestressed Concrete Pile
Composed of the following sections:
1-60' Length with 20' Tapered section and 40' constant section
1-21'-11½" Length of constant section
Test Pile No. T-10

Length-in-place	81'-11½"
Penetration below ground	79'-11½"
Cut-off elevation	1095.41
Tip elevation	1013.45

Pile Driven May 4 and 8, 1959 & tested July 6, 7, 8 and August 1, 2 and 3, 1959.

DRIVING DATA

Pile Driven with McKiernan-Terry DE-30 Diesel Hammer

Rated Energy	18,000 Ft.-Lbs.
Weight of Ram	2,800 Lbs.
Weight of Driving Cap	900 Lbs.
Approximate weight of pile at final penetration	16,900 Lbs.
Driving time	2 hours-10 minutes

72nd STREET SOUTH OF GROVER ST.
OMAHA, NEBRASKA

Fig. 8. 14 inch octagonal prestressed concrete piling load test.

strands. Forms are simple bottom pallets and side forms with crossties at the top. The piles are reinforced spirally with ½ in. rod bent into a continuous spiral circling the outside of the pretensioning strands. The majority of pretensioning bearing piles are either square or octagonal in cross-section. The small diameter piles are solid and the larger diameter piles have cored holes.

The recent joint AASHO-PCI committee has developed pile standards which are of square and octagonal cross-sections and of dimensions from 10 in. to 24 in. in 2 in. increments. Driving tips can be blunt or tapered to suit driving conditions. The build up of piles is normally accomplished with a mild steel reinforced connection. The AASHO-PCI standards state "The minimum area of reinforcing steel shall be 1½ of the gross cross-section of concrete. Placement of bars shall be in a symmetrical pattern of not less than four bars." A typical pile build up may consist of 8 number 6 bars 3 ft. long embedded into the head of the pile. (fig. 3). If the pile has to be lengthened one method is to break away the top of the pile exposing the tie bars and spirals. The splice bars are then embedded and the additional length of pile is poured in place on top of the original pile.

Tops of all concrete piles are usually protected during driving by some type of cushioning between the top of the pile and the hammer. This assures distribution of the hammer blow and tends to alleviate shock and eccentric driving. The most common type of cushioning is a 6 in. layer of softwood or a 2 in. layer of laminated plywood. These cushions must be replaced during driving before they become so compressed that they do not supply the desired cushioning effect. Sometimes two or more cushions are needed in driving one long pile.

The typical pile cross-sections are relatively simple to manufacture. Prestressed concrete piles can be used when combined with splicing techniques in any lengths and in most all ground conditions.

GENERAL NOTES

Prestressed concrete piles have several properties which make them preferable to other types for many applications. They withstand severe driving conditions without cracking or spalling. A non-prestressed concrete pile will crack as it recoils from a heavy hammer blow. When the hammer strikes, its force creates compression in the pile which causes an elastic shortening. As soon as the energy of the hammer is expended and there is no force to maintain the elastic shortening, the pile springs back to its original length. By the time it reaches its original length it has gathered momentum causing further elongation which in turn creates tensile stresses in the pile. Under a heavy hammer blow these tensile stresses cause cracks, and under repeated blows spalling develops. Prestressed concrete piles are fabricated with sufficient compressive stress to off-set the tensile stresses caused by recoil. Experience indicates that an initial compressive stress of between 600 psi and 700 psi is normally sufficient for this purpose. Where especially severe driving conditions exist the magnitude of the prestress force should be increased. Final determination of the amount required is a matter of trial and experience with the existing driving conditions.

Taking into account the tensile stresses obtained during driving, it is the writers opinion that the diesel hammers which have recently

become popular for pile driving, are far superior to other types of driving equipment previously used, that is, the steam and the drop hammer.

The diesel hammer delivers blows at a far higher rate of speed than a steam or drop hammer; it might be that this high speed of blows does not allow the concrete to reach the tensile stresses caused by the elongation due to recoil before the next blow hits the pile.

It is also the writers opinion, that, there are cases where considerably less than 600 psi compressive stress is desirable in piling. Piling for support of sewer pipe, support of building slab, or small commercial buildings can be made 6, 8, or 10″ square with as little as 200 psi compressive stress, assuming that the pile lengths are very short and that the stresses due to driving are light.

Prestressed concrete piles are easy to lift and transport. Their compressive stress enables them to resist a large bending moment, so that they can be lifted with simple one or two point picks where reinforced concrete piles of the same dimensions require complicated rigging from multi-point picks. Transportation by pole trailer in lengths of up to 90 ft. has been accomplished and they also have been transported by rail car where the stresses due to humping of the cars are quite intense and might cause serious damage to other products. see fig. 9.

Prestressed concrete piles are durable because they are crackless. They are seldom cracked by driving or handling and even if they should be, their compressive stress would keep the cracks tightly closed once the pile is in place. Because of this a prestressed concrete pile has a much higher moment of inertia than the reinforced concrete pile of the same dimensions. In the prestressed pile the entire cross-sectional area of the concrete contributes to the moment of inertia, while in the reinforced concrete pile the concrete in the tensile side has no function and only reinforcing contributes to the moment of inertia. This additional moment of inertia is important in piles whose capacities are determined by their slenderness ratio.

Reference

Preston, H. Kent, **Practical Prestressed Concrete,** McGraw-Hill Book Co.

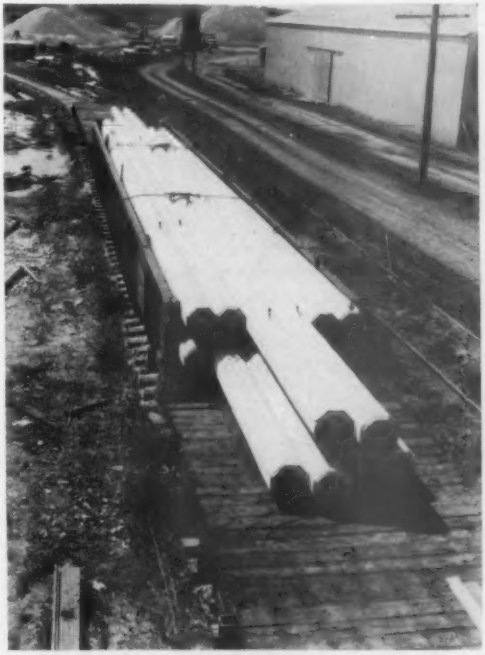


Fig. 9. Typical load of piles on a rail car showing ease of handling and transporting prestressed piling.

Recent Developments In The Automatic Manufacture Of Prestressed Members In The USSR

by Prof. V. V. Mikhailov*

Presented at the 6th Annual Prestressed Concrete Institute Convention

In the USSR mechanization and automation in prestressed concrete construction manufacture is now being introduced on an ever growing scale. Hundreds and hundreds of small and large prestressing plants are equipped with mechanized and automatic machines for placing and stressing reinforcement and concrete casting.

The degree of mechanization in production effects the final cost of the prestressed products and the distribution of the net cost between the phases of production.

Table No. 1 gives the variation in cost of the elements of expenditure in percentage of the total cost as the degree of mechanization varies from high to low.

The relative cost of materials per concrete member in this way increases from 45% to 62.5% and the relative labor cost per member decreases when high mechanization replaces low mechanization. Other elements vary but slightly.

It is very clear that the cost of precast concrete in mechanized factories is lower. Analysing this data, especially for highly mechanized plants, we see that a great price savings can be obtained by economy in materials, because these represent the major part of cost per member. This is achieved by using high strength concrete and steel, and economical member profiles such as thin-walled and spatial structures.

The application of prestressed concrete in construction is an important technical innovation; however it involves greater labor. For example, labor required for different types of prestressed construction takes from 80 to 500 manhours per ton of prestressed steel reinforcement instead of 50 manhours per ton for ordinary bar reinforcement. The precast pre-

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Table No. 1

No.	Elements of Expenditure	Relative cost in % by mechanization level		
		High	Average	Low
1	Materials	62.5	56.9	45.00
2	Wages including overhead	11.88	16.70	32.60
3	Repair and amortization	10.85	12.60	8.60
4	Other expenditures	14.47	13.80	13.80
—		100%	100%	100%

stressed construction cost noticeably decreases with the use of stressing and vibrocasting machines.

Methods of continuous concrete prestressing and casting by vibropunching, vibrorolling, vibropressing, etc., open up new horizons in this field. The seven year program of industrial development in the USSR provides for an approximate 25% lowering of construction costs. First we reconsidered the profiles and sizes of present prestressed concrete members. Then we studied production methods, while using the most mechanized methods of steel stressing and concrete casting. This is being carried out in the field of large-scale dwelling construction.

At the 1957 World Conference of Prestressed Concrete in San Francisco we had an opportunity to inform American engineers about the continuous prestressing method in use at large factories in the USSR.

Certain of our plants are now in the process of mechanization. Automation of many processes is also under way.

Our work in this field is not only to introduce present advanced technology but to do the research necessary to develop new automatic methods for precast prestressed construction manufacture of tomorrow.

In the course of the forthcoming seven year period (Seven Year Plan), mechanization and automation of all plants, small and large, will be completed.

By placing the prestressed wire so that it passes through the concrete core of the member in the direction of the tension stresses, it becomes possible to thin the member web and wall, to lower the stressed reinforcement, and to eliminate nonstressed bars.

Continuous prestressing makes it possible to use continuous wire feed ing, stressing, and placing in one complete mechanized and automatic process in manufacturing all types of members.

A good example is the construction of a one story industrial building. The precast components were a ribbed floor panel 12 m. long and 3 m. wide; "I" beams 24 m. long (see World Conference report); a track girder 12 m. long, for a 30 ton crane, and flat cross-ribbed wall panels 12 m. long.

The twelve meter long track girder (fig. 1) is 1.4 m. high and is assembled from two channel like pieces. All girder section zones have continuous prestressed reinforcement—longitudinal in the top and bottom flanges and transverse in the channel web.

The longitudinal prestressed wires do not reach the beam supports, but at definite point change their direction and bend at a right angle to beam axis and prestress the girder web. This heavy prestressing of the girder web permits a decrease in thickness to 5 cm.

The continuous reinforcement lends itself to modern technology and can be accomplished quickly and easily.

Photo (fig. 1) shows the 12 m. long girder in testing, and photo (fig. 2) this girder at flexural failure.

The girder test showed that crack occurring at the load Q = 120 t. The girder failed at the load Q = 320 t., equal to designed load. Because of the web prestressing, the inclined cracks were noticed late in the test and did not cause the girder rupture.

At present a number of continuous stressing machines are in production and development. The most extensively used machines CPV-3, DN5, MP, ARM, and DN7, are known to engineers who partici-

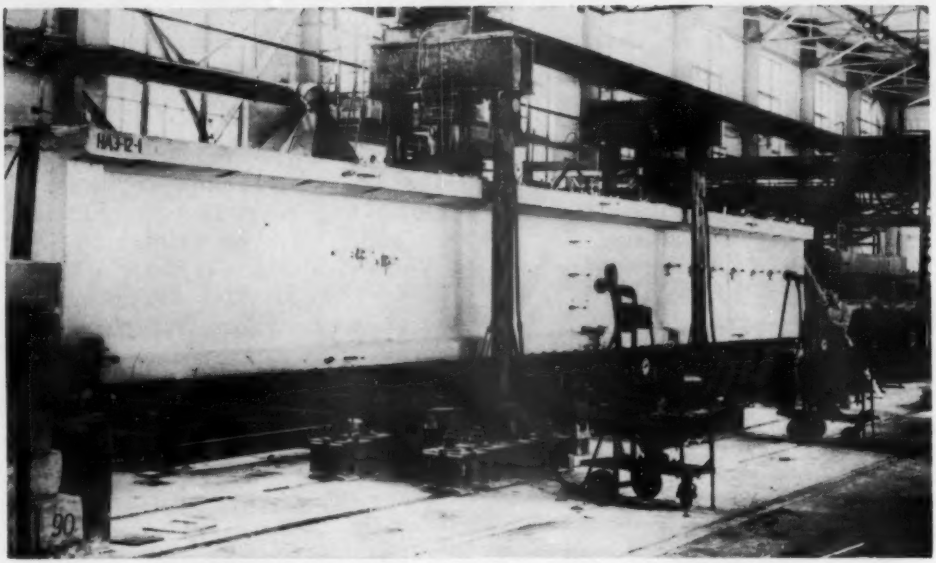

Fig. 1 Crane girder 12 m. long with a 30 ton capacity being test loaded.

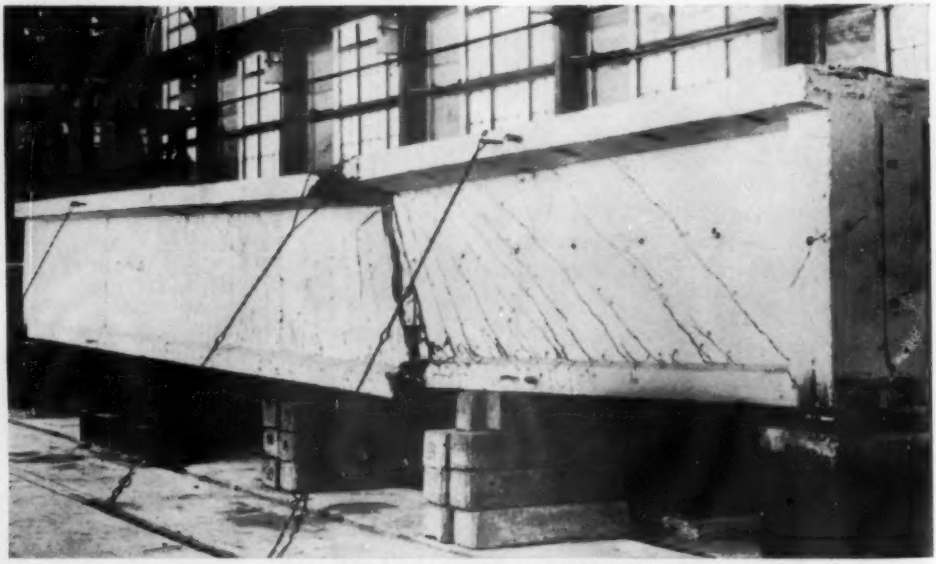

Fig. 2 Crane girder after test to failure.

pated in the San Francisco Conference, and who have visited the USSR. These machines all simultaneously distribute and stress two wires 3 mm., or one 5 mm. in diameter. The speed of wire distribution is about 0.3 to 0.5 meters per second. Recently the production of continuous stressing machines has been increased considerably by using electro-thermo-mechanical methods of prestressing.

One of the marked shortcomings of all continuous stressing machines is that winding requires high quality stress relieved wire. The lowest elongation of this tensioned wire at rupture must be not less than 3%. If the wire used is of poorer quality failure often occurs and a long piece of winded stressed wire loses its prestressing and slides down. For this reason, in the case of multi-row winding and at a steep bend, we must decrease the prestressing from 0.65 to 0.55 and even to 0.50 ultimate strength.

The electro-thermo-mechanical prestressing method can be described as follows:

The wire is prestressed to 0.20-0.30% of its ultimate strength. It then receives additional electrothermal heating and elongation while moving in the process. The heating is done electrically and as a result the continuous wire is heated to 250° or 300° C. This heated and stressed wire winds on mould pins. When the stressed winded wire cools an additional stress of about 0.45-0.35% ultimate strength is produced. The total prestressing of the wire, after initial mechanical stressing and following stressing after cooling, is the required 0.65 ultimate strength.

Quick wire heating up to 300°C. improves the plastic quality of the wire, as does stress relieving widely applied in the USA and other foreign countries. This doubles the productivity of continuous stressing machines by allowing them to wind a greater number of wires, or permitting the use of larger diameter wires. Faster stressing table rotation also increases productivity and makes it possible to wind two 4 mm. wires, or one 7.8 mm strand instead of two 3 mm. wires.

The oil on the wire surface burns up during heating, thus improving the wire anchorage in concrete. Special automatic regulation of current strength and tension makes it possible to standardize the temperature of heating and level of stressing, independent of winding wire length. The wire distributing mechanism of the winding machine is generally the first contact of the electric chain; the second contact must be the last pin or anchorage on the movable mould on which continuous wire winds. During winding the next pin always serves as the second contact which continues throughout the system.

The use of electrothermal continuous stressing excludes wire tearing and the stoppage of stressing machines. Electro-thermo-stressing is a definite improvement in the method of continuous prestressing. The labor consumed by continuous prestressing now is 20-25 manhours per ton of winded wire, considerably less than in other prestressing methods. Low labor expenditure and higher productivity permit the development of automatic wire distribution, placing, fixing and stressing processes. A light stressed framework at a very low price is thereby obtained.

Vibropunching

A new factor lowering the cost of prestressed concrete construction,

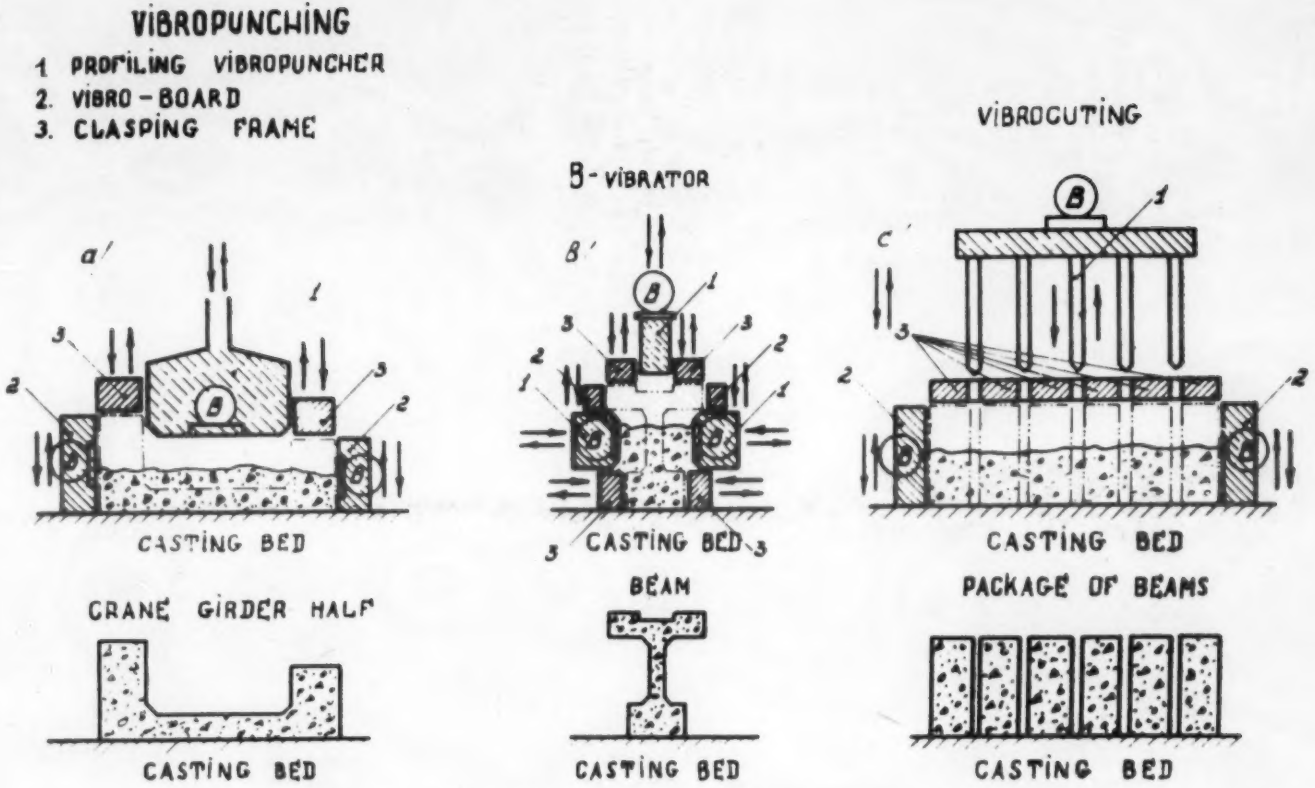

Fig. 3 Typical vibropunching processes.

is the innovation of concrete vibropunching during casting. The free vibration method used in casting of concrete members also gives good results, but here the concrete density is attained only from the constructions own weight. Concrete mixed by vibration becomes plastic and spreads till its surface flattens –the stiffer the mixture the more time required for casting.

During vibration the stiff mixture acquires plasticity and fluidity. It is therefore possible to direct, move, replace and densify the mixture, in any, even very complicated profiles, by force. Vibropunchers or even boards in close contact with the concrete mixture can be used for vibropunching casting.

On completion of the above described mechanized casting, and after vibration ceases, the mixture sets, and becomes strong and hard.

The profile and density of a member is obtained by the vibropuncher (1) (fig. 3) moving in the direction of the concrete mixture simultaneously with the application of a pressure of about 200 g/cm^2.

The exact profile print on the surface of a cast member is obtained from the same profile on the vibropuncher. The vibropunching method of member casting provides for quick vibropuncher and form board removal without injury to the concrete. Therefore mould separation during vibropunching is an important link in the member casting process. The chief parts of the vibropunching machine are:

1. Profiling vibropuncher
2. Vibroboard
3. Clasping frame

All vibropunching machine parts must be mobile and their lifting is usually done by hydraulic jacks, since vibropunching requires application of high stresses. Fig. 4 shows vibropunching methods for various members. Individual members (fig. 4a, b) can be cast by vibropunching and member sets (fig. 4c) employ vibrocutting. When vibrocutting, vibroknives are attached to the vibropuncher. The process of vibropunching consists of the following operations: (see fig. 4)

1. Board lowering
2. Pouring of concrete
3. Clasping frame lowering
4. Vibro puncher driving into the concrete mixture, and distributing it into the casting member volume. This operation coincides with vibration.
5. Switching off the vibrators of the vibropuncher
6. Board lifting
7. Vibropuncher lifting
8. Clasping frame lifting

Through special valves, compressed air is fed under the vibropuncher to speed its separation from the concrete. As a result the bond break is quick and without member damage.

Vibropunching has been applied, but without complete mechanization, in our production of concrete dwelling members. We are also seeking to improve automatic and more complicated vibropunching machines for large scale concrete member production.

A very interesting example of a vibropunching machine is the automatic vibropunching line for producing prestressed cross ribbed sound-proof composite floor panels. They are built at the Concrete and Reinforced Concrete Research Institute of the Academy of Construction and Architecture of the USSR, Moscow. The cross-ribbed prestressed composite panel with soundproof plates and floor parquet slab is the most economical floor construction for our precast dwelling buildings.

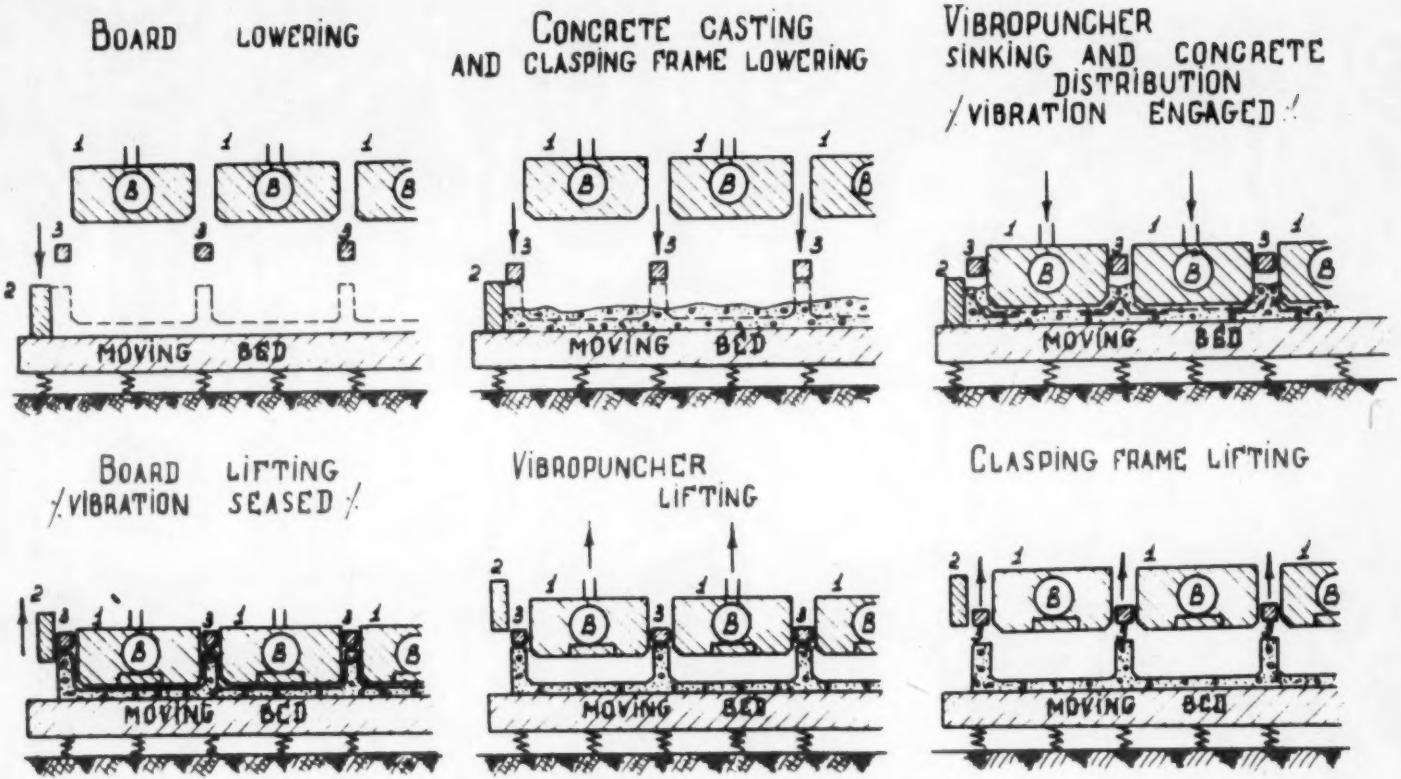

Fig. 4 Construction stages of a vibropunched floor panel.

PRESTRESSED COMPOSITE SOUNDPROOF CROSS RIBBED FLOOR PANEL

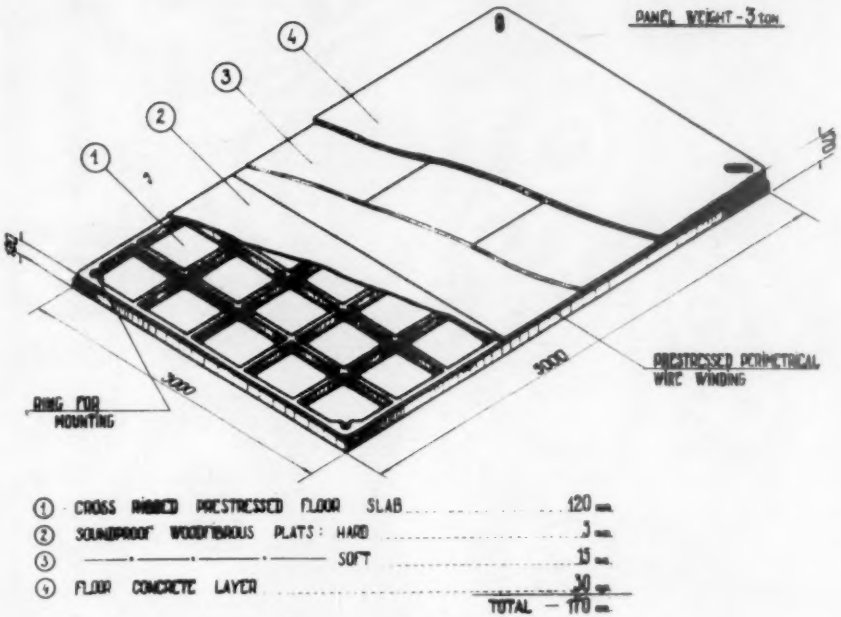

Fig. 5

It requires 4.5 cm of dense concrete and 1 kg of reinforcing steel per square meter. (fig. 5)

Three machines, a stressing table DN5 or MP, a concrete copying feeder, and a vibropuncher machine are used on the production line.

The production set up of continuous prestressing machines is well known so it will not be reported here.

The concrete copying feeder is a new machine with the following parts: (see fig. 6) self-propelling bunker (1), dosage box (2), lowering distributing knife (3), self-propelled gantry (4) with moving wide conveyor belt attached (5) and with restricting boards (6). The copying feeder delivers the required quantity of concrete on the gantry belt as needed, or in square layers. Next it throws the concrete down onto the moving bed and distributes it in the designed dimensions in the necessary sequence, or in a flat layer convenient for vibropunching.

The copying feeder functions as follows: Bottomless bunker filled with concrete slips over the open dosage box (2), which fills up as the bunker moves forward and backward. It is then vibrated to provide a very dense concrete box loading. At 180° the dosage box turns, discharging the concrete mixture on the conveyor belt (5). The gantry (4) begins to move forward and back, simultaneously the distributing knife (3) lowers to the first, second and third level in consecutive order and spreads the concrete mixture in a square flat layer on the conveyer

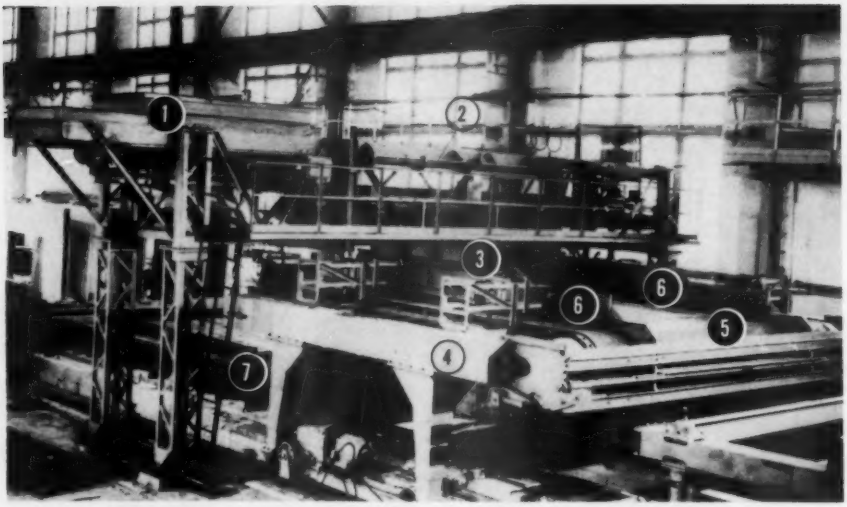

Fig. 6 Concrete copying feeder.

belt (5). Two cycles of knife lowering are sufficient to distribute the concrete mixture in exact square layers. The gantry then stops over the movable bed, the belt begins to roll and the concrete mixture falls down on the bed. The gantry again begins to move and the concrete mixture is spread on the bed surface directly in the required dimension of the future slab. The knife profile can make whatever needed surface relief is required in a transversal direction to the concrete which has been thrown down. The necessary concrete relief in a longitudinal direction is accomplished by stopping the gantry.

The complete cycle of copying feeder requires 7½ minutes; however simultaneous operations reduces the exact cycle of automatic line work to approximately 4 minutes.

The vibropunching machine (fig. 7)

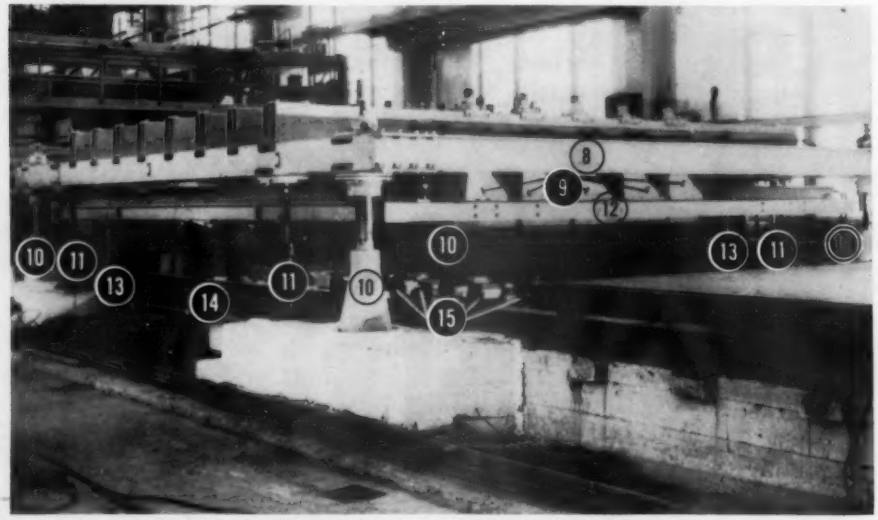

Fig. 7 Cross-ribbed floor panel vibropunching machine.

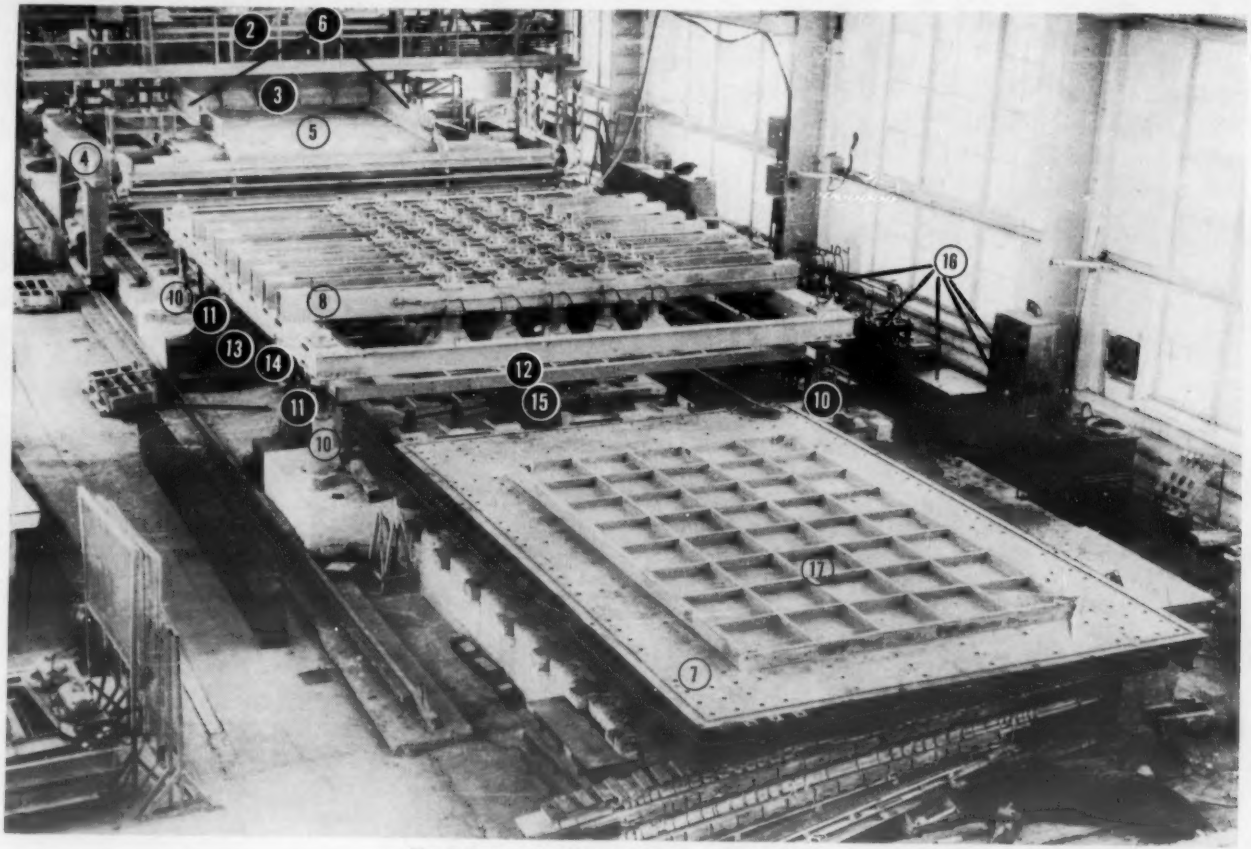

Fig. 8 Automatic vibropunching production line.

has the following parts:
1. Profiling vibropuncher with main frame (8) and 60x60 cm. chests, (9) raises and lowers, with the aid of 4 guides (10), and jacks (11)
2. Clasping frame (12) lifted by jacks (13)
3. Lifting rail (14) to lower the bed (7) with concrete mixture and place the bed on the shock absorber springs (15)
4. Self-propelled rolling brush to clean the concrete mixture and dust accidentally bonded to the chests and clasping frame.

In the described automatic line the boards are attached to the moving bed and for this reason there are several changes. The work of the automatic line is done in the following procedure: fig. 8.
1. Rolling the moving bed (7) under vibropunching machine then lowering and placing it on the springs (15)
2. Lowering the clasping frame (12)
3. Vibropuncher (8) lowers, thereby causing the chests to press and vibrate the concrete mixture. The hard set concrete mixture moves under chest pressure, fills the ribs, reaches the clasping frame and forms the desired unit dimension.
4. Chest vibration ends
5. Pressed air under chest is fed with simultaneous jacking
6. Clasping frame is lifted
7. Movable bed (7) is lifted to rail level
8. Finished panel (17) with bed is moved and next operation starts.

All these operations consume 3-4 minutes, with only a foreman in attendance. The copying feeder work and the simultaneous work of automatic line machines, plus some extra time, makes the production sequence about 6 minutes. In this way the automatic line can produce 600,000 sq.m. (6,400,000 sq. ft.) of floor panels annually.

Another example of effective mechanized work is the production set-up of continuous prestressing plants with DN7 and DW57 machines. The plant is for industrial member production and produces the following: roof slabs 3x6 and 3x12 m.; I beams 6, 12 and 24 m. long; crane girders 6 and 12 m. long (30 ton capacity); and wall panels

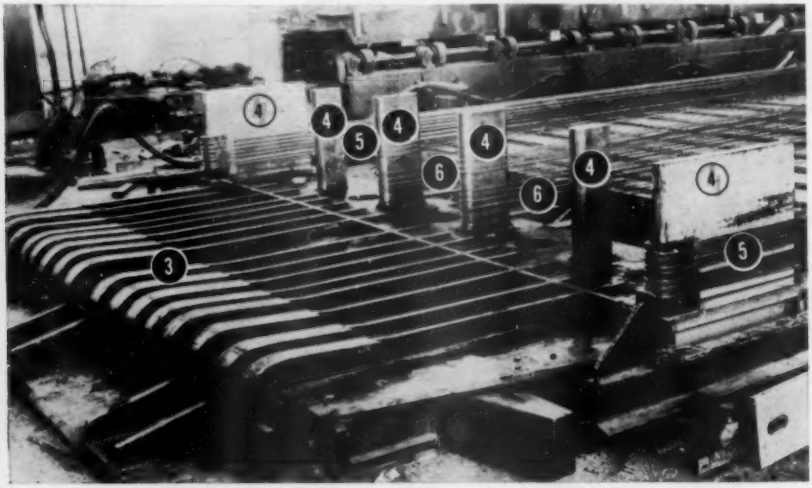

Fig. 9 Anchorage end of DN7 production line.

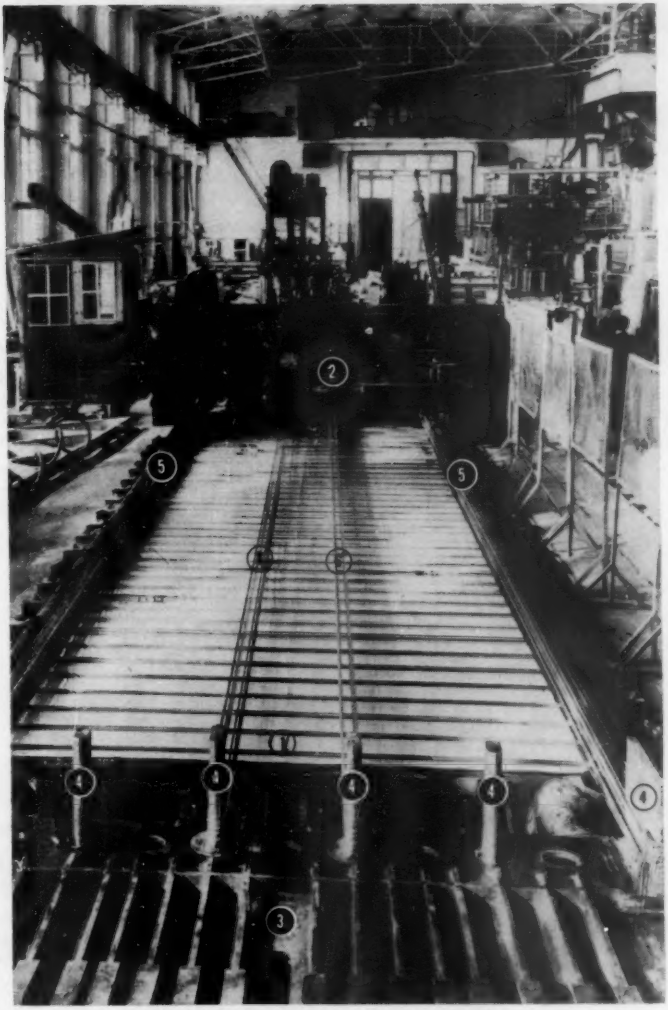

Fig. 10 Production line with a DN7 machine.

3x12 m. The plant bed (figs. 9 and 10 is made from precast prestressed concrete blocks 3.2 x 1.2 m. and is 67 m. long. Each bed end is constructed like a large concrete anchor with a movable steel anchorage (3). Movable pins (4) are arranged at each side. Continuous reinforcement is wound longitudinally and transversally in the proper sequence around these anchorages.

Two machines work on the bed: a continuous stressing DN7 machine for stressing reinforcement in two axis and a vibropunching DW57 machine for casting and punching the concrete member. The product sequence is as follows. The vibropunching machine, self-propelled into a new position, begins to cast the member.

Casting members are distributed close to one another giving good bed usage. For example, in crane girder production two girder halves are close to each other, completely

filling the casting bed, with extremely little space between members. This space is used to cut the stressed wires between the separate girder halves.

Vibropunching machines supplied with spare parts are operated by 2 men and can be reassembled to punch various members quickly by changing the vibropuncher and boards.

Three types of factories producing continuous prestressed precast concrete members for industrial buildings with production capacities of 200, 400 and 600 thousand sq.m. of one story buildings yearly are now in design. Preliminary calculation establishes that buildings assembled from continuous prestressed precast construction will, when compared to present prestressed methods yield the following economies: 20% concrete, 25% high tensile steel, 30% labor expenditures and 25% cost. Steel economy resulting from the introduction of continuous prestressing and vibropunching at casting during the first year and a half make up in full the steel required for the machines. The outlay for factory buildings with equipment was also refunded within two years.

Our first and most important task is quickly to improve and master all new machines at prestressing plants, test them on automatic lines, and prepare them for use everywhere in large-scale production.

"Extrusion of Prestressed Concrete."

by Harry H. Edwards*

Presented at the 6th Annual Convention of the Prestressed Concrete Institute

In the mass-production of prestressed concrete there are two principal problems. One is material handling and the other is our system of forming.

The two are pretty much tied together. The system of forming that is used will dictate very largely the acceptable material handling system that can be used. It also determines importantly the cost of material handling.

The prestressed industry as a production industry is very young. It is old in theory but in production, it is only ten years old. We have gone through a great many stages in the short period of ten years, both in materials handling and in forming systems.

As would be expected, our first forming attempts were in movable forms, which necessitated a lay-out covering quite a large area. This dictated a type of material handling system and a production lay-out largely in the open, to suit the forming system.

As the other systems have come along, we have not replaced our system of movable forms. We have merely separated it.

The next step was to go to a system of forming utilizing a fixed set of forms. Examples of fixed forms are double Tee's, channel sections, bridge beams, etc. It is possible to produce all of these members in a fixed form where, you pour it into a form and take members out of the form, working on a daily cycle without disturbing the form.

That was a great step forward. However, what was introduced was a system of forming an entirely new product. Therefore, we now have two systems of forming that look like they are well established within the industry—the movable form and the fixed form. In spite of its drawbacks the movable form is an essential part, and it looks like we will have it with us always, particularly for certain large shapes, and custom work.

In our relatively new fixed form system—we have its limitation of producing only certain shapes.

The prestressed industry, in order to completely cover the building field, had to find additional shapes, shapes that would fit into other purposes for other functions. Demands that aren't being completely filled by the double Tee's, the Tee joists or panel sections.

The industry soon became aware that a prestressed hollow core flat slab was needed.

Of course hollow core slabs have been made for several years now. These hollow core slabs were made by many different systems—some in

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fixed forms—some in movable forms. The big variation is in the coring system. There have been cores made with round paper tubes, square tubes, plastic tubes. There are also pneumatic inflated tubes. Some of the void materials have been left in the forms, others are withdrawn.

These products were preceeded by flexi-core which is a reinforced concrete product. It has attained a surprisingly large productive capacity, and large sales all over the country. The prestressed concrete industry could not help but look at the very fine success that had been attained by this reinforced product, and naturally evolved a system to do it with prestressing.

Many producers today make such a product, usually by the wet method, where wet concrete is poured into a fixed mold. They have made a product that is far superior to a reinforced concrete hollow core slab. It has taken care of the problem of heavier loads, and also the problems of camber and deflection.

The success of the industry with the hollow core slab, has convinced various people that have been working on automation to do the same thing.

Fig. 1 Slip form extrusion machine casting a keystone joist in an inverted position. The members are cast so dry that they can be walked on upon leaving the machine.

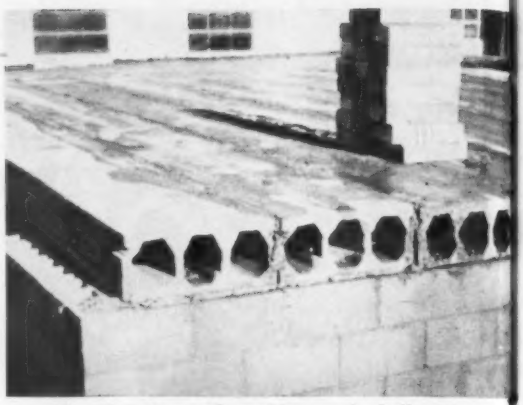


Fig. 2 A type of machine made hollow core slab currently in production.

The results so far have attained savings in costs when compared to reinforced concrete hollow core slabs, even with the difficulties of handling the tubes.

In spite of those difficulties, the hollow core industry has got a very firm foothold in the construction market in the last two or three years—on a cost and quality product basis.

It looks now as though the industry will take another step, which is coming very fast, that is to produce the same product completely automatically. I suppose, however, five years from now we will be looking upon our crude attempts and say, well, we sure missed the boat back there. But it looks very attractive today because we have available several processes by which we can make a continuous product of a hollow core cross-section, using a more efficient concrete mix and making a stronger product. The systems actually produce a much larger yearly production volume, with less capital investment.

On that basis it is easy to predict that the industry is going to go very

strong for the hollow core slab with its various systems of automation.

Now, the title of our general discussion here is Extrusion. You might say that concrete can be extruded—at least, under certain conditions. The layman thinks—and we all think—of squeezing toothpaste out on your brush—as an example of extrusion. You can put a pressure on a plastic mix that will force the product through a die. If it were stiff enough, as the toothpaste is, you could have a hollow core extrusion in there. Spaghetti is an example of what you could consider an extruded product. There are in addition a great many products that are extruded.

If you take a pile of rock and sand and mix them up and put that in a container with a die on one side of it and put a lot of pressure on it, you will not squeeze it out through the die, or through the orifice. You will break the container before you will squeeze it through.

Of course, concrete is a plastic to a degree. Our cement gives us a plastic jell. The problem presented to the industry, is to take these rather hash mixtures of low slump and be able to extrude them, or make them float through a particular shape that you are interested in.

Due to these difficulties, we have several processes in which hollow core slabs are made, and of course, the easiest one to visualize is a simple slip-forming system.

We are all familiar with the slip forming that has been going on for years in silo work. The concrete mix is poured into a vertical form which is moved up either continuously or intermittently. Here the mix conrol is not too great, you are not producing a void, and the pour is vertical.

Slip forming can also be done in a horizontal position, where you pour into a form which moves along the bed. This has been done in road work very successfully, largely where the edges are slip formed and the top is screened off.

A core section can be placed into any mold, if the concrete is fluid enough and vibrated, it will flow underneath the core, which then can be pulled along. This is one of the systems that is being developed. It is strictly a slip-forming system.

Then there are many modifications to this system, with variations in the feeding, or forcing, and procedures for making the concrete plastic.

In the very dry mixes we can actually create a conditioned mix to float. When you pick up the mix in your hand, like a block mix it looks and feels perfectly dry. However if vibration is used to produce a degree of plasticity that is combined with pressure, the mix can be made to float.

We are all familiar with the work that vibration plays in reinforced concrete work. We have found with heavy vibration it is possible to get a combination of extrusion and slip-forming. However, the industry likes the word "extrusion," so I suppose we will have the name of extrusion with us from now on for all of our systems.

As regards to the shapes that can be produced, it is possible to extrude or slip-form a great many shapes. Right now the industry is in a stage of finding out the economics of the various shapes.

Several different shapes have been produced by continuous slip-forming or extrusion. The I-section, the inverted T, the channel section, flat slabs, and Keystone joists have all been produced. Those are all relatively easy since they have no

Fig. 3 Pilot plant in Tennessee with casting machine in operation. Electrically operated machine runs right on the concrete bed using the prestressed strands as guides.

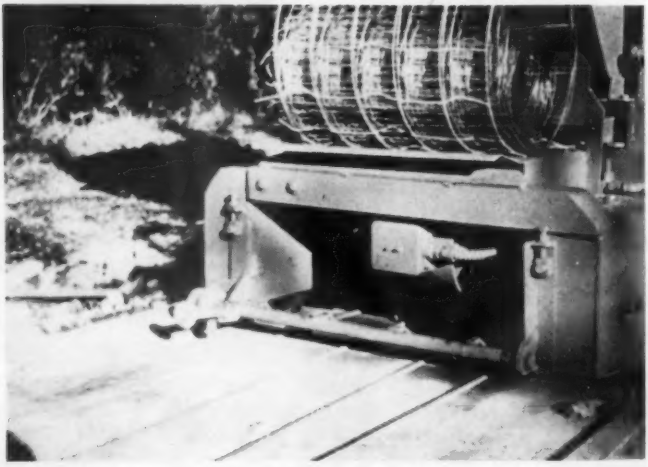


Fig. 4 Close-up of the wire side of the machine showing the guide which steers the machine along the strand.

Fig. 5 Load test for completed slab.

voids in them.

So the industry right now, due to economic reasons, is sort of letting these simple shapes sit on the side a little bit and has gone to the more difficult task of extruding a hollow core product.

Of course, the reason is that there is a terrific market for the very efficient hollow core section. It can be produced at an extremely low cost, and therefore a great deal of effort has gone into the hollow core plank.

The accompanying photographs were supplied by David Dodd, who is a pioneer in the field of heavy vibration on a dry mix. These will help to illustrate the current status of the industry.

In summary I believe the automatic production of hollow core slabs has progressed very fast and believe it will continue to grow as its acceptance and use is expanded. I believe it will be followed by the same type of machine producing a solid plank. This machine, however, wouldn't travel at 3½ feet per minute but it will produce economical plank at 7 feet per minute.

Fig. 7 Production line set up in Syracuse, New York plant.

Fig. 6 Machine in operation laying the cored slabs on a simple concrete pallet.

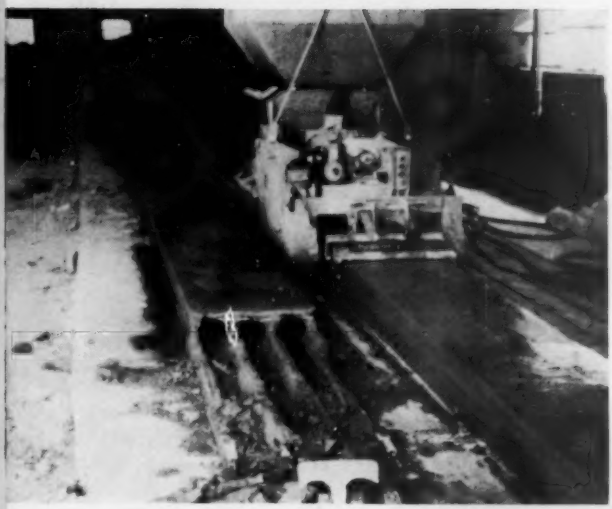


Fig. 8 Machine in operation on the production line.

Fig. 9 Machine finishing up on one line and being moved to the next by a gantry crane.

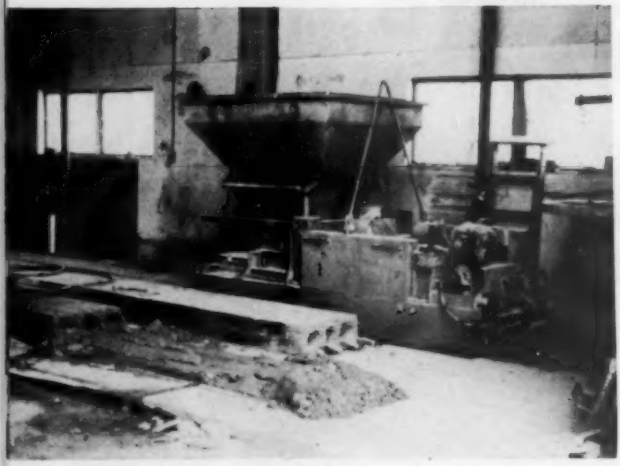


Fig. 10 Machine starting up third line. Note the simple strand anchorage required because of the low prestress force and low overturning moment.

Fig. 11 Cut-off saw that cuts the slabs to required length. Usually cut the day following casting and then removed from the bed.

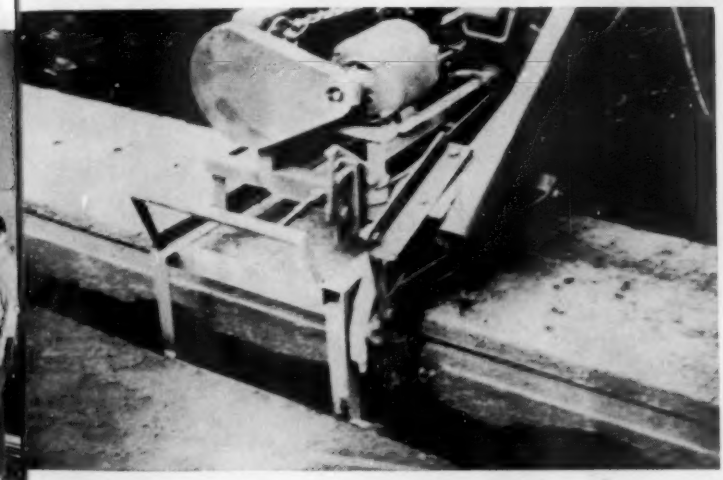


Fig. 12 Close-up of the cutting operation.

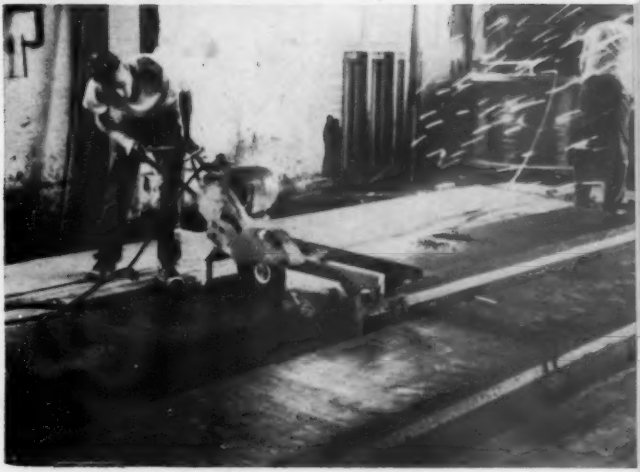
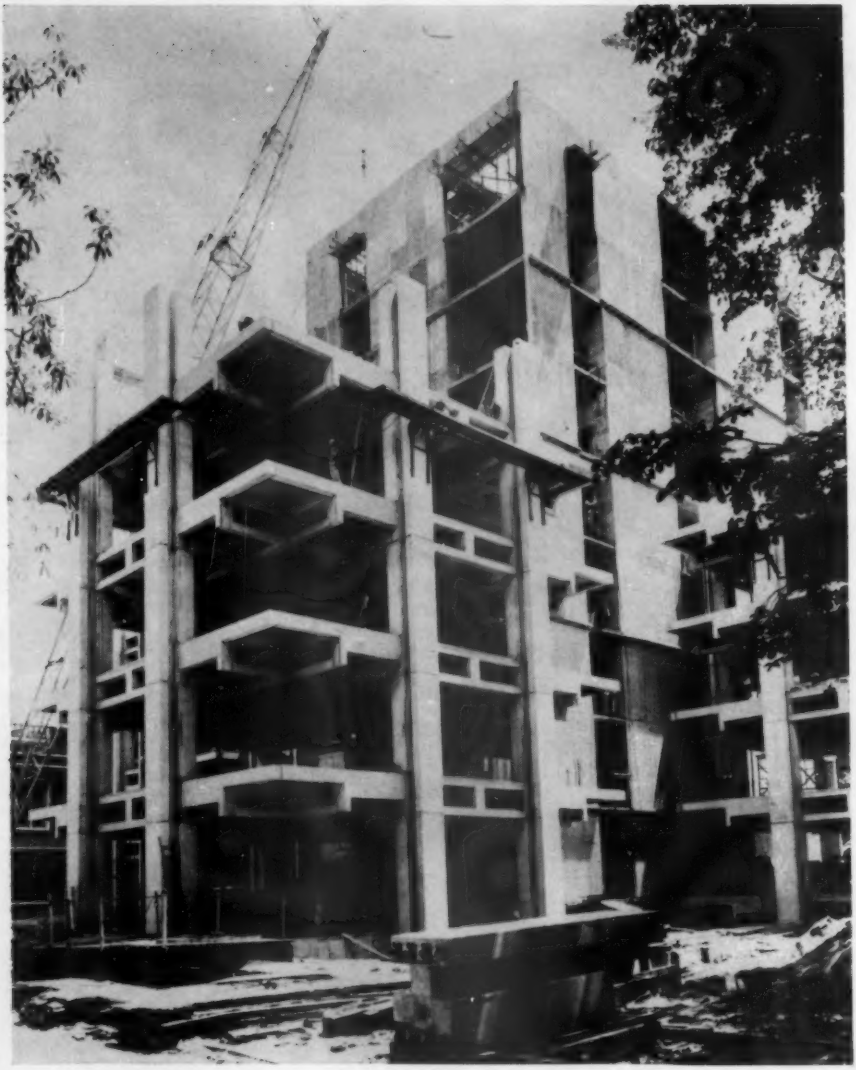


Fig. 13 Cutting operation in the plant employing a double cut.



Credits: Architect: Louis I. Kahn, F.A.I.A.; Structural Engineers: Keast & Hood; Consulting Structural Engineer: Dr. August E. Komendant; Prefabricator: Altantic Prestressed Concrete Co.; General Contractor: Joseph R. Farrell, Inc. Photos: Stressteel Corp.

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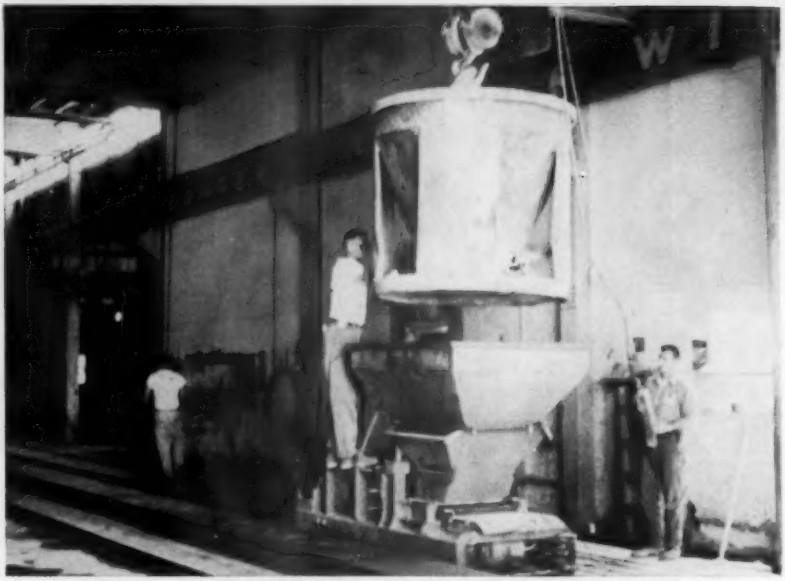

Fig. 14 Plant loading operation using a dry mix. The machine can be stopped for loading or it can be loaded while in operation.

Fig. 15 Load test conducted in Syracuse.

Electrothermic Method of Pretensioning Bar Reinforcement of Precast Reinforced Concrete

by B. G. Skramtaev*

Presented at the Sixth Annual Convention of the Prestressed Concrete Institute

Precast standard prestressed reinforced concrete structural units are being manufactured in the USSR in large quantities on continuous production and conveyor system lines. The tensioning of bar reinforcement by means of jacks requires large expenditures on equipment and much labor. Therefore, under conditions of mechanical tensioning it is very difficult to organize mass production of prestressed reinforced concrete structural units.

Recently the situation has radically changed due to broad application in the USSR of the electrothermic method of prestressing the reinforcing bars.

This method employs heating of the reinforcement outside the molds and subsequent cooling in the moulds. This procedure was worked out in the Mosgorispolkom Municipal Research Institute of Reinforced Concrete Products.

The theory of this method consists of prefabricating reinforcing bars which are shorter than the distance between mould anchorages (fig. 1). After being heated by electric current the bars extend and become longer than the distance between anchorages and are, therefore, quite easily laid into moulds. In cooling the bars try to shorten and thus are tensioned to the given degree.

The degree of prestressing depends upon the original difference between the length of the bar end anchors and the distance between the supporting surfaces of mould anchorages. Consequently the exactness of prestressing depends on the accuracy of these dimensions.

In order to provide accurate dimensions, the anchors on the reinforcing bars as well as the anchorages of the moulds are placed with the help of special gauges. However, even in this case some deviations from the given degree of elongation are inevitable. Therefore it became necessary to investigate the effect produced by allowable tolerances of the degree of prestressing.

In determining the allowable deviation from the required initial prestressing use was made of the specific features of the bar reinforcement which distinguish it from high tensile wire.

In structures with high tensile wire the appearance of cracks is followed by increasing deflections which may be explained by a very low percentage of reinforcement. Consequently in members, where there is danger of corrosion this should be taken into consideration.

Bar reinforcement used in pre-

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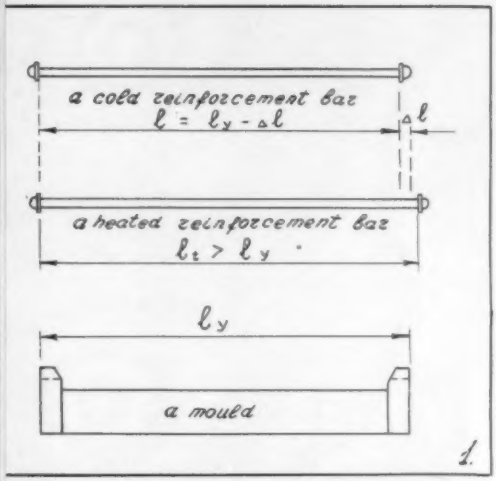


Fig. 1—Simplified diagram of electrothermic prestressing.

stressed concrete has relatively large diameters (12-30 mm) (.47-1.18 in) and considering the danger of corrosion, it is as susceptible as any kind of hot rolled steel used in reinforced concrete without prestressing. Moreover in constructions with bar reinforcement of 30XC2S type the accumulation of deflections after the appearance of cracks increases with less intensity than does high tensile wire (fig. 2). Therefore in most structures prestressed with bar reinforcement, cracks up to 0.2 mm wide are allowed to appear during loading. In such cases it is only necessary to ensure sufficient rigidity of construction and to restrict the expansion of cracks; which is why prestressing is used.

In many mass produced reinforced concrete units these requirements are satisfied during the initial prestressing of 30-40 kg/mm² (42,500-57,000 psi). At the same time the initial prestressing of reinforcement may be brought up to 90 per cent of the yield point which for strengthened steel of grade 25G2S and 30XG2S means 50-54 kg/mm² (71,-000-77,000 psi). Thus it becomes possible to create conditions of large allowed steel stress deviations which are not usual for prestressed reinforced concrete. Stresses within these allowances are ensured by using special gauges: e.g. for ribbed floor slabs the accepted initial prestressing force is no less than 36 and no more than 52 kg/mm².

The checking of the degree of initial prestressing was carried out on a large number of bars. This was done by means of tensometers placed at the ends of reinforcing bars not subjected to heating. Mass testing carried out by many plants has shown that with the accepted methods of using accurate gauges for the placing of anchors on the bar ends and for fixing the mould anchorages; the degree of initial prestressing never exceeds predetermined limits.

At the present moment the electrothermic method of pretensioning the bar reinforcement is in use in many Soviet plants. In Moscow alone over 3 million square meters (32 million sq. ft.) of precast floor slabs are being produced per annum by means of this method.

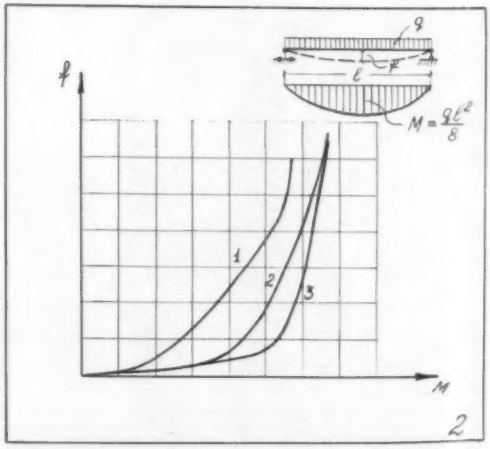


Fig. 2—Graph of deflection versus moment for bars (3) and strands (1).

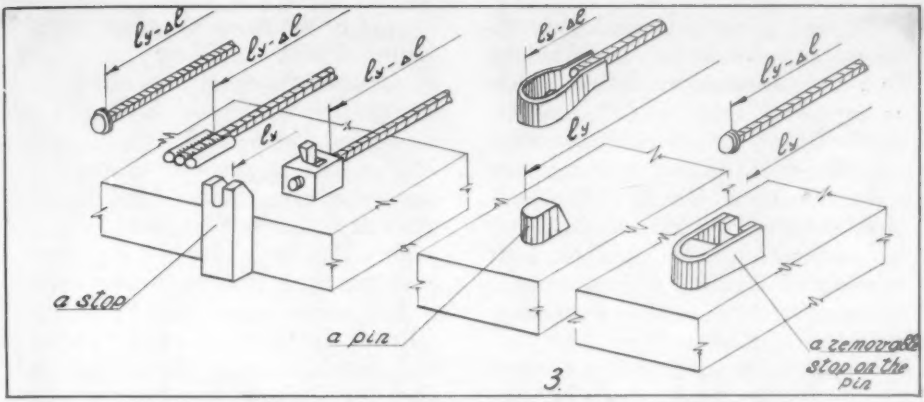


Fig. 3—Five different types of anchors used at the ends of the prestress bars.

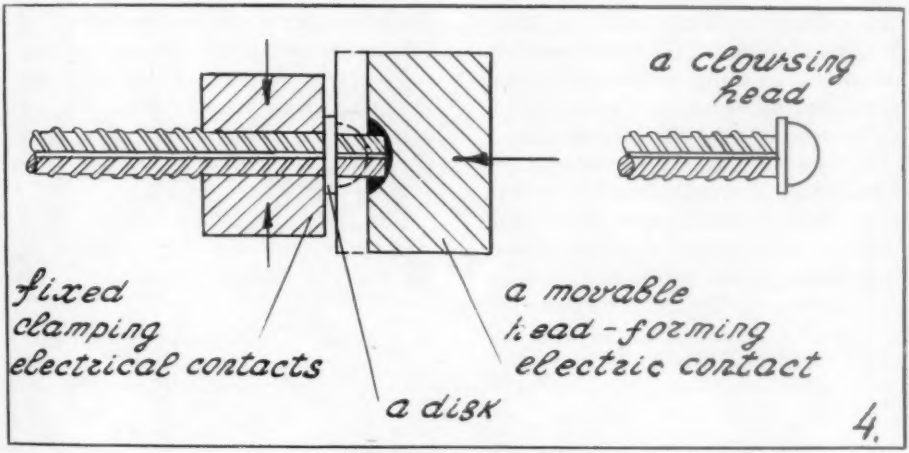


Fig. 4—Bar heading machine.

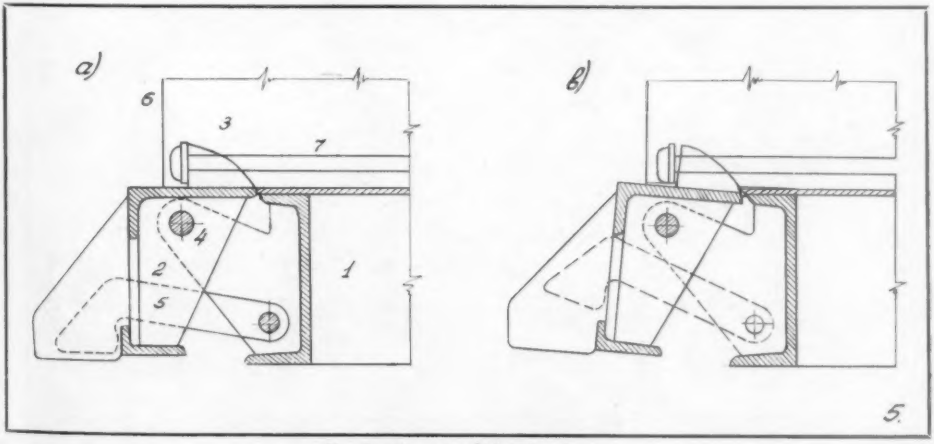


Fig. 5—Hinged self locking form.

Depending on local conditions different anchors at the ends of the bars and different mould anchorages are being used (fig. 3). Anchors in the form of heads and fixed anchorages of the fork type are most commonly used. Heads at the ends of reinforcing bars are accomplished by machines usually used for butt welding reinforcing, with additional clamping and up-setting pneumatic cylinders (fig. 4).

During a 7-hour shift two workers can head, by means of a machine, from 800 to 1000 bars. Before heading the bar is usually capped with a washer which insures uniform distribution of pressure on the mould anchorages. In order to have adequate strength the heading has to be exactly symmetrical to the bar axis.

Where the need for reinforcing bars is comparatively low the anchor is made by welding to each end of the bar two short pieces of reinforcing waste, 40-45 mm long, and 14-18 mm in diameter. At some plants equipped with mould-cars and dowels for continuous wire winding, the bar anchors are made in the form of welded loops made of flat steel. This makes it possible to use the same cars for winding high tensile wires and tensioned bar steel.

The "Barricade" factory in Leningrad uses multi-purpose stock clamps of the wedge type. (fig. 3)

In all the cases described above, with moulds having fixed anchors, after the concrete has acquired the predetermined strength of 150-200 kg cm 2(2100-2800 psi) the ends of the bars are cut off thus transferring the prestressing to the concrete.

There are some other solutions where additional economy is achieved by eliminating the reinforcement cutting.

Here are two examples.

For manufacturing cored floor slabs one plant uses trays which take up the prestressing stress and also

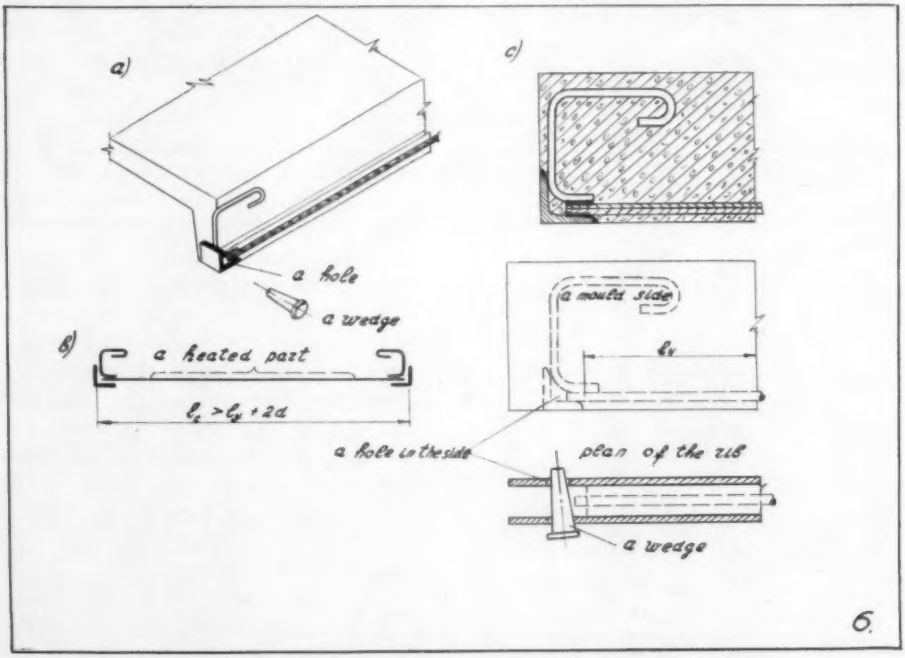


Fig. 6—Use weld plates to fix prestressed reinforcing to the form.

allow the upper part of the mould to be taken off directly after placing and compacting the concrete.

The tray is provided with thrusts in the form of segments which are placed in the concrete (fig. 5). When the concrete has acquired sufficient strength the tray with the precast product is put on the formwork post and the projections are rested upon special thrusts. Due to the weight of the tray with the slab, the projections open and the movable end of the tray rotates in relation to the fixed part of the tray. The thrust then comes out from the body of concrete and the slab tears away from the tray as well as from the fixed thrusts placed at the opposite end of the tray. The slab is then taken from the tray by a crane.

Another example takes advantage of the insert details used for fixing the slabs to roof trusses or beams. When there is no prestressing, longitudinal reinforcement and additional anchoring bars are welded to the angles (fig. 6-a).

When the members are prestressed with reinforcing bars the insert angles are used as anchors to fix the bars temporary on the mould without any projection of prestress reinforcing from the concrete. To accomplish this a hoop is attached to each end of the prestress rods. (fig. 6-b).

Therefore a space is formed between the angle, the reinforcement and the hoop bar and into this space a wedge is inserted which transfers the prestress force to the form while the concrete is acquiring the predetermined release strength. These wedges are inserted after placing the heated reinforcement into the mould.

Upon cooling and shorting of the reinforcement the supporting angles set against the wedges and prestress forces develop in the reinforcement. The tension forces are transferred through the wedges of the steel mould. After the concrete has acquired its predetermined strength the wedges are knocked out and the prestressing is transferred to the con-

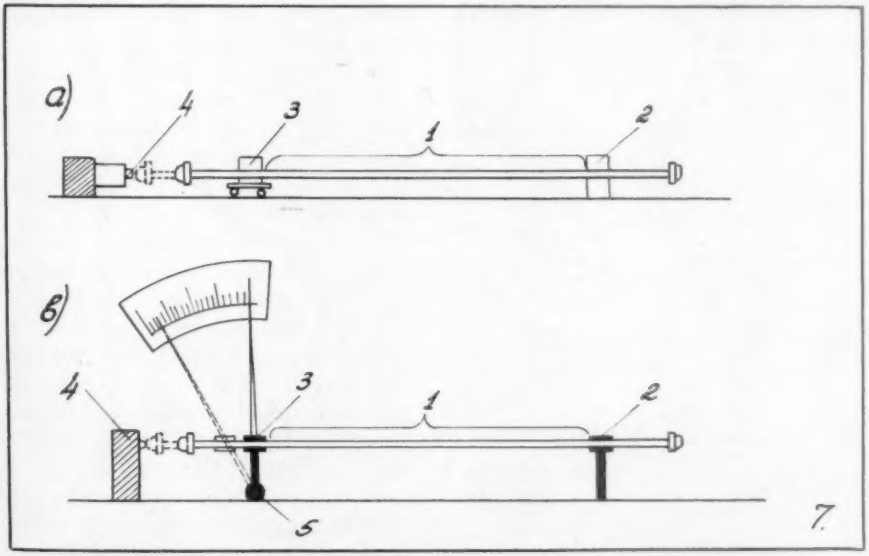


Fig. 7—Methods used to turn off the current used in heating the prestressed reinforcing.

crete.

Reinforcing bars are heated on special installations equipped with transformers which are similar to those used for electric welding. Heating is accomplished by connecting contracts to the bars in two points at the center of the bar, some distance from the ends. During the process the central part of the bar is heated up to 250-350°C (420-600°F) while its ends remain cold. This permits the placement of the heated bars in the mould without any special equipment. Tight contact of the electrode with the bar is ensured by a pneumatic press.

Research has shown that the heating of the above mentioned types of steel to temperatures of 460-500° C (780-870°F.) does not reduce their strength. While heating is being carried out, special measures are taken to prevent temperature rise above the predetermined degree. To accomplish this one contact on the bar remains fixed during heating while the other one moves together with the elongation of the bar (fig. 7). When the predetermined elongation is reached a switch cuts off the current and the heating automatically stops. Some heating installations have in addition a device with a pointer indicating the elongation of the bars during heating. One installation simultaneously heats from 2 to 6 bars depending on the type of reinforced concrete construction that is being produced. Heating usually lasts from 2 to 4 minutes.

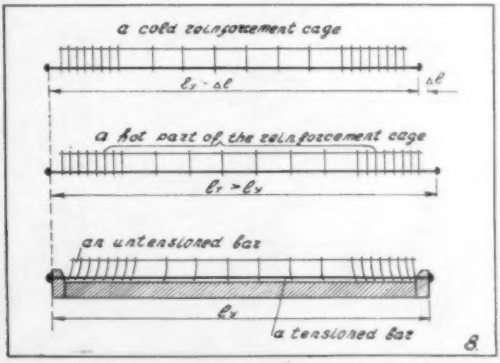


Fig. 8—Diagram showing effect of electrothermic prestressing on reinforcing cage.

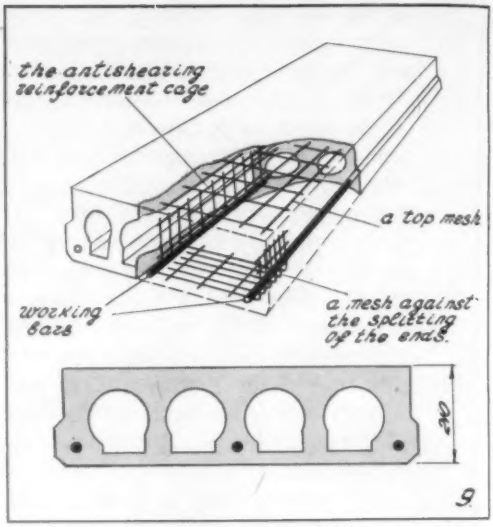


Fig. 9—Precast cored slab.

Heated bars are then placed into the mould thrusts where they cool and acquire the predetermined initial prestressing. Under usual workshop conditions the cooling from 250-350°C to 80-90°C lasts 12-15 minutes. It may be accelerated by means of artificial cooling. Concreting is allowed when the temperature is lower than 90°C. (130°F.).

Heating may be applied not only to separate reinforcing bars but also to bars constituting elements of welded frames. Here part of the auxiliary reinforcement is being heated but during cooling only the prestressing steel having end anchors is stressed. Due to the flexibility of the assembly disturbance of the welds never occurs (fig. 8).

The application of bar reinforcement to mass produced prestressed concrete structural units has its own

structural and technological characteristics which must be taken into consideration when organizing the production processes. Let us take some typical examples.

In the USSR the most commonly used structural element for housing is the cored floor slab. The electrothermic method of tensioning as previously described is used in manufacturing these members.

Formerly the practice was to make cavities of a circular cross section; recently the cavity is increased by welding channels to the bottom of the void-forming plungers. The height of the slab is also reduced by 10 mm at the expense of the thickness of the upper flange. This decreased the consumption of concrete by 18 per cent (fig. 9). Adequate placing and compacting of the concrete is ensured by internal vibration of the plungers. The prestressed reinforcement is 12 and 14 mm in diameter and is placed into every second or third rib of the slab.

When transferring the prestress force to the concrete the sectional reinforcement compresses the con-

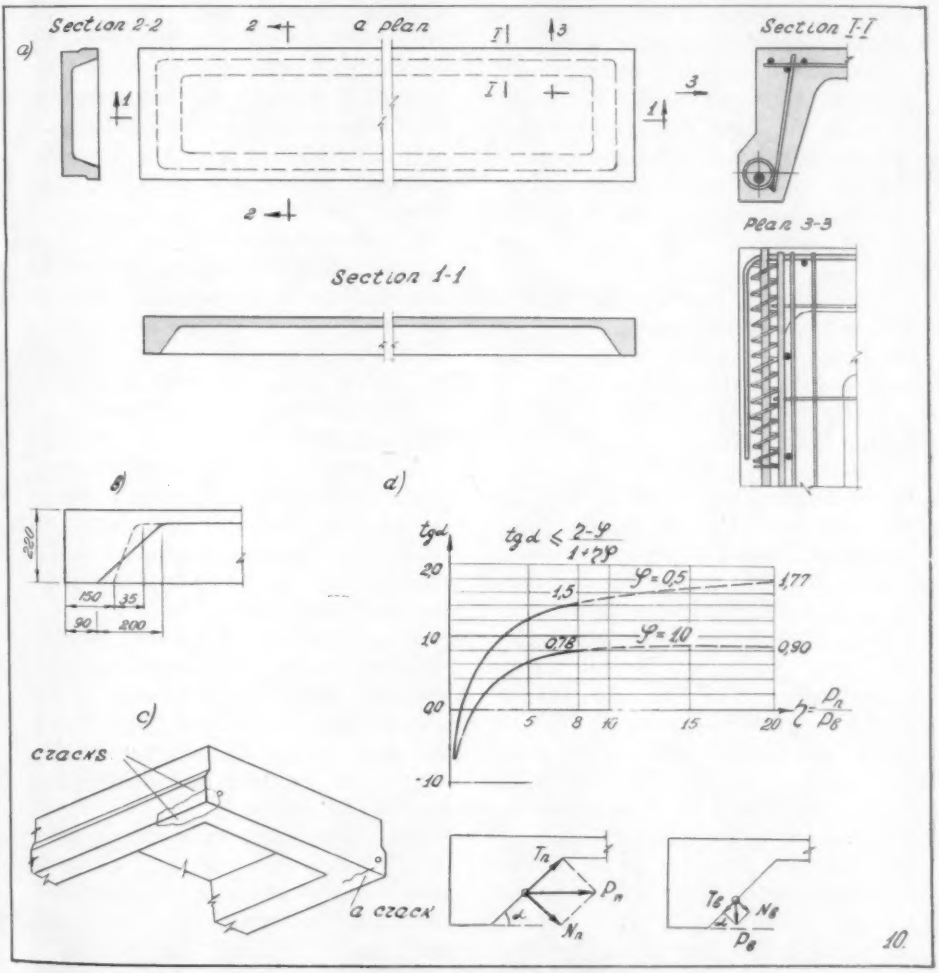


Fig. 10—Precast ribbed floor slabs.

crete and, at the same time, tries to split it. If special measures are not taken, cracks may appear and develop on the end faces of the slabs.

In order to prevent cracking in cored slabs at their end faces, welded meshes of cold-drawn wire 3 or 4 mm in diameter are laid. These meshes are only 0.3 m long and have produced excellent results.

As another example let us take the ribbed floor slab. Dwelling houses as well as civil buildings obviously require slabs with smooth surfaces for the ceilings and floors. Such requirement is completely satisfied by cored slabs. As to the floors between basements and ground floors here smooth surface is needed only for the floor. For such floors it is quite possible to use not the cored but the ribbed slabs which reduce the consumption of concrete by 28-38 per cent and which cost 25 per cent less.

Due to prestressing the ribbed slab may have the same height as the cored one (220-210 mm) which ensures interchangeability.

Prestressed ribbed floor slabs are trough-like in shape with ribs located along the perimeter (fig. 10). Each longitudinal rib is reinforced with prestressed reinforcement comprised of 1 bar 14 mm in diameter, grade 30XC2S, and with non-prestressed reinforcement consisting of 1 bar 10 mm in diameter, grade 25C2S.

End ribs are of the same height as the longitudinal ones in order to ensure the direct transfer of pressure from walls without additional work on the job. But, as it is well known, the transverse rib after transfer of the prestress force is pressed into the mould projection which prevents the free shortening of the concrete, and complicates the striking of the formwork. And indeed in the process of striking the forms of the first experimental slabs, cracking almost always appeared at the intersections of longitudinal and transverse ribs.

Investigations have shown that the striking of slabs after the transfer of prestressing to the concrete is effected by the angle of slope of the internal facet of the transverse rib.

When this internal facet has but a very slight slope prestressing assists in releasing the product from the form.

The value of the critical angle which is called "the angle of self-striking" depends upon the ratio of prestressing force to the product's weight and upon the coefficient of friction of concrete against the steel form. This coefficient includes not only friction but also the possible cohesion of concrete with the surface of the form.

Calculations and experiments have shown that for ribbed slabs the angle of slope of the internal facet of the end rib should be about 45°. After the forms with the correct angle have been manufactured the striking goes on unimpeded and no cracks appear.

To prevent the splitting of concrete in narrow ribs, spirals about 50 mm in dia. 300 mm long, of wire 3 mm in dia., spaced at 25-30 mm, are put on the ends of the prestressed reinforcement.

In addition to cored and ribbed slabs prestressed floor beams and girders are also being produced. These girders are 6 m long with a 160 x 600 mm cross section and are designed for a uniformly distributed load of 3000 kg/m.

By substituting prestressed steel 30X2S for ordinary reinforcing steel which has yield points of 25-40 kg/mm^2 the consumption of steel for each girder is more than halved.

The main reinforcing of a girder

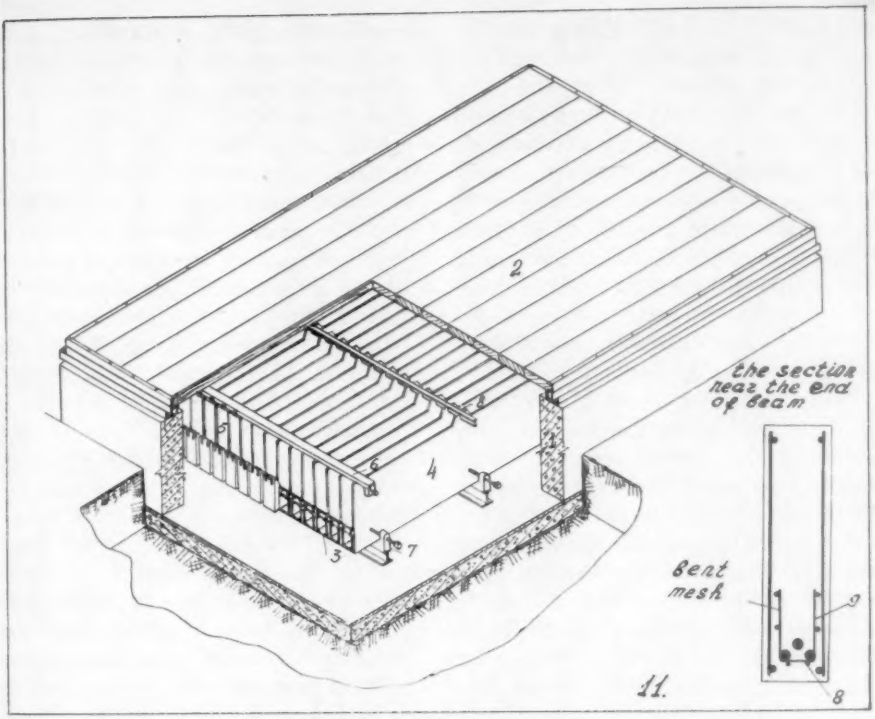


Fig. 11—Multiple girder form.

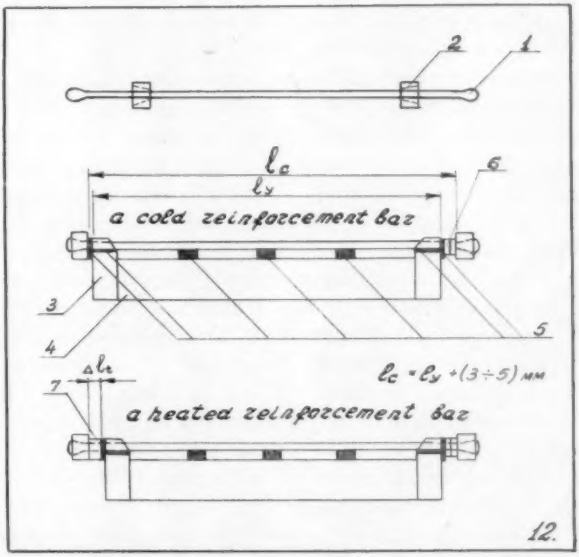


Fig. 12—Electrothermic prestressing within the moulds (forms).

consists of 3 bars, 14 mm in dia., placed at different levels within the girder (fig. 11).

In order to avoid splitting, a small turned-up mesh is used at the ends of the girders.

The girders are manufactured in a multiple form consisting of 10-20 sections and placed in steam curing chambers. During the curing time the prestress forces are resisted by the steel construction constituting the lower part of the form. (fig. 11)

This report has dealt chiefly with the electrothermic method of prestressing bar reinforcement with heating accomplished before placing in the mould. However, heating reinforcing in the moulds has also been studied. The first experiments with reinforcement heated in the moulds were carried out in the USSR, in Cheliabinsk, by the Ural branch of the USSR Academy of Construction and Architecture. Later on in Moscow the method of tensioning with reinforcement heated outside the form was worked out, and this method proved itself as being simpler as well as cheaper and is now used everywhere. However in some cases heating within the moulds appears to be quite efficient. When using this method the bars are made somewhat longer than the distance between thrusts for easy placement. (fig. 12). The possible inequality in the length of the bars is then eliminated by the driving in of thin "levelling" forked washers between the anchors of the bar and the thrust. After this operation the current is switched on, the bars freely elongate and their anchors move off from the thrusts of the mould. When the bars have sufficiently elongated a "measured" washer is inserted whose thickness corresponds to the predetermined extent of the reinforcement elongation. After placing the "measured" washer the current is cut off and the reinforcement becomes stressed upon cooling. It is obvious that the ends of the reinforcing bars which are in the thrusts have to be isolated from the steel form. No contact between the central part of bars and the form is allowed either.

The drawback of this method lies in the increase of the labor for the manufacture of the structural details. This increase is due to the application of the portable contacts, insulation of thrusts, and use of additional washers etc.

In mass production of prestressed details of comparatively small dimensions (6-8 m long) it is advisable to heat reinforcing bars aside and then carry them hot to the form.

In case of long details, piles, masts etc., when to carry long hot bars presents some difficulty the heating of bars on forms may be quite successfully applied.

Up to the middle of 1959 the electrothermic method of tensioning reinforcement of prestressed concrete structures has been used for manufacturing slabs of a length up to 7 m.

Further development of this method has shown that good results may be attained for longer structures. At present in the USSR various structures are being manufactured: trusses and roof beams, large-size slabs, piles, posts and others by tensioning of the reinforcement by the electrothermic method. The tensioning is accomplished by the wires cooling and is completed before the placing and hardening of the concrete mix. In all cases heating of the reinforcement outside of the moulds has been found cheaper and reduces labor requirements. As an illustration three examples of prestressed structures are used: i.e. roof beams of industrial buildings with a 18 m

span, roof trusses with a 12 m span, and large size double I-section slabs.

The 18 *m span roof beams* have a curved upper edge approaching a parabola. This curve creates conditions for ideal moment resistance. The beam is of I-section with the main stressed reinforcement in the lower flange plus some in the web. The rest of the reinforcement which has not been prestressed, is made in the form of welded screens and cages. (Figures 13, 14, and 15)

Two beams are manufactured simultaneously, on a stand, in a horizontal position. The prestress forces are resisted by anchor plates in the reinforced concrete beam placed below the mould.

A removable steel insert is placed between the manufactured beams for performing simultaneous electrothermic heating of two 14 mm bars. The installation consists of two contacts with eccentric springs and insulated rolls, which prevent sagging of the bars. The transformers are placed between the mould pairs. From the side of one of the contacts is a terminal switch, automatically cutting off the current after the required lengthening of the reinforcing bars during their heating.

Roof trusses 12 m long are designed for one-story industrial buildings with a column spacing of 12x24 m. They support beams which span 24 m and are spaced every 6 m. As the production of these roof trusses was started after mastering the manufacture of the roof beams, several improvements were made in their production.

The bottom flange of the truss is reinforced by 10 stressed bars of low alloy steel with a 22 mm. dia. The other parts of the truss are compressed under load and are not prestressed; they are reinforced with welded three-dimensional cages. (Figures 16, 17, and 18)

The moulds are designed so that two trusses are manufactured simultaneously. The tension forces of the reinforcing bars are resisted by the fixed spacer placed between the lower flanges of the two moulded trusses. This steel spacer is part of the mould, which is freely supported on the bottom supports. During steam curing the mould is heated simultaneously with the tensioned bars and tensioning is not weakened as it is with concrete supports.

The middle spacer also serves as part of the moulds as it forms the lower surface of the truss. Due to this the bending forces in the spacer are very low.

The outer side of each half of the mould is made as an integral member, and is pulled away from the hardened concrete by four screw-type hand-wheels.

The spacer and outer sides of the mould have steam ducts for contact heating of the concrete plus the entire mould is placed in a steam chamber.

Large-size double-T-section panels. When these structures were being made we took into account available experience in U.S.A. where double T-section slabs are usually made 1.2 m (4 ft.) wide. (Figures 19, 20, and 21)

Taking into account the advantage of employing large-size precast members and the availability of transport and assembly machinery in the USSR, double T-section panels are made 3.0 m (10 ft) wide, 9, 12 and 15 m (30, 39, 49 ft) long and 45 and 60 cm (18 and 24 in) high.

The long panels as are the previously mentioned beams and roof trusses are made of concrete of a strength of 350 kg/cm^2 (5,000 psi). The strength of the concrete during transfer is not less than 225 kg/cm^2

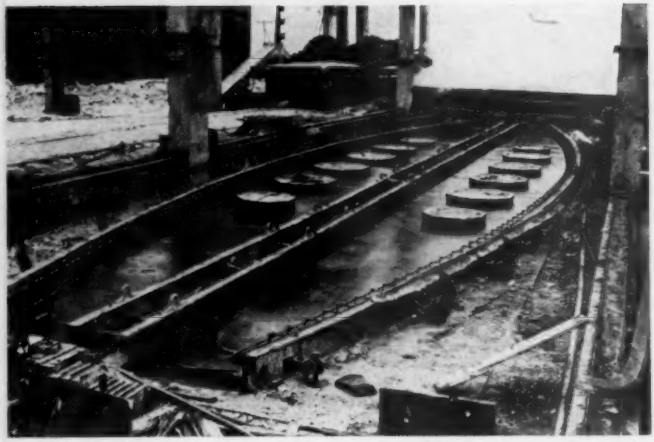


Fig. 13—Forms for manufacturing curved roof beams.

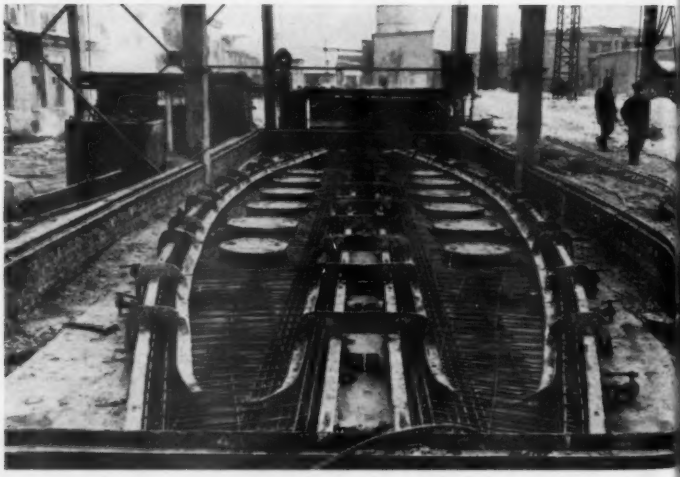


Fig. 14—Reinforcing in place in roof beam forms.

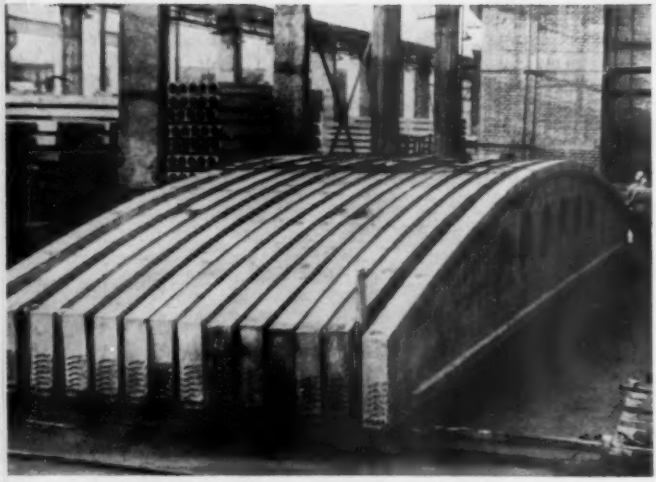


Fig. 15—Completed roof beams.

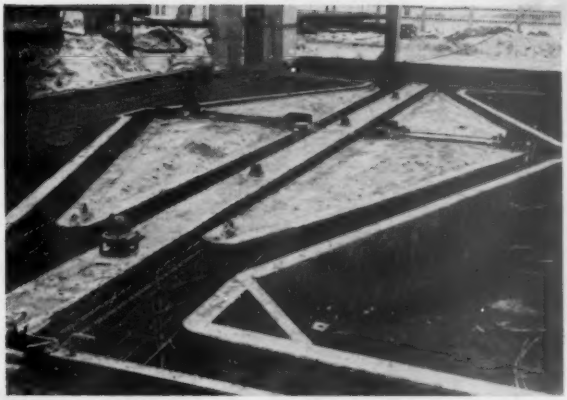


Fig. 16—Roof truss forms.

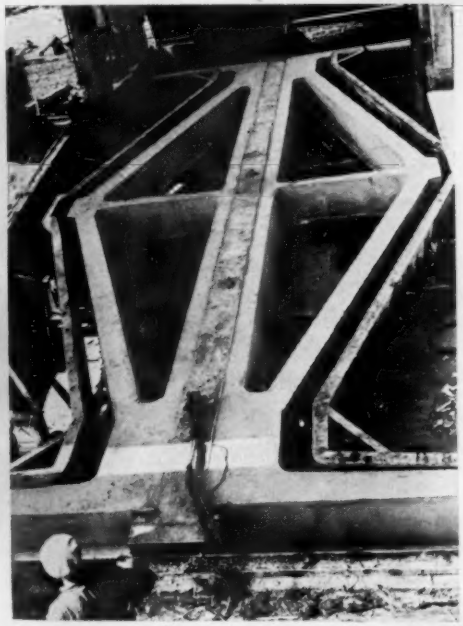


Fig. 17—Completed trusses being stripped from the forms. Note wheel on screw device for pulling top form away from concrete.

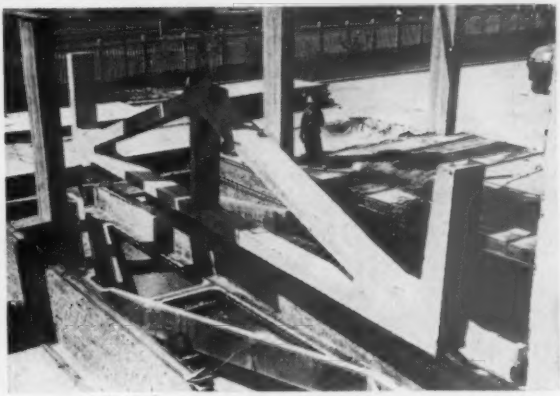


Fig. 18—Completed truss being lifted from form.

Fig. 19—Completed double tee's. An end diaphragm is cast with the tee to stiffen the slab edges.

(3,200 psi).

The average concrete thickness is 8 cm for a length of 9 m, and 10 cm for 12 and 15 meters. The panels are designed for live loads of 350 and 600 kg/cm^2 (123 psf).

Taking into account the width of the panel, low transverse ribs are cast at the ends to prevent damage to the thin slab during transportation and erection.

The stressed working reinforcement of each of the two ribs of the panel consists of 4-8 bars of low alloy steel, 14 mm in dia.

The panels are manufactured in separate steel integral moulds, which also resist the stressing forces. The moulds are placed in steam chambers. The entire cycle of panel manufacture, including steam curing does not exceed 20 hours.

We consider it expedient to have a special mould for each standard size, without using inserts, as done in the U.S.A. This is explained by our guaranteed sale of products and obtained economy.

The machinery for electric heating of the reinforcing bars is portable so that one heating installation is used for several moulds.

The hot bars are carried from the heating installation to the mould by 3 or 4 workers, two holding the bars by hooks. Heating and placing of one pair of reinforcing bars into the mould takes about 4 minutes.

In our building practice we also post-tension the reinforcing bars in set concrete. The reinforcement is laid, without coating, in opened or closed channels.

The main characteristic of the Soviet method of electrothermic post-tensioning is that tensioning without coating has been worked out and is widely used as distinct from post-tensioning the reinforcement with coating as suggested by Germany and USA but not used in practice.

In conclusion we should like to say that no new method of prestressing was ever mastered with such speed by numerous enterprises and in construction jobs in different parts of the Soviet Union, as that of electrothermic tensioning during the last 18 months.

In the USSR electrical heating is now beginning to be used in pretensioning high tensile wire also.

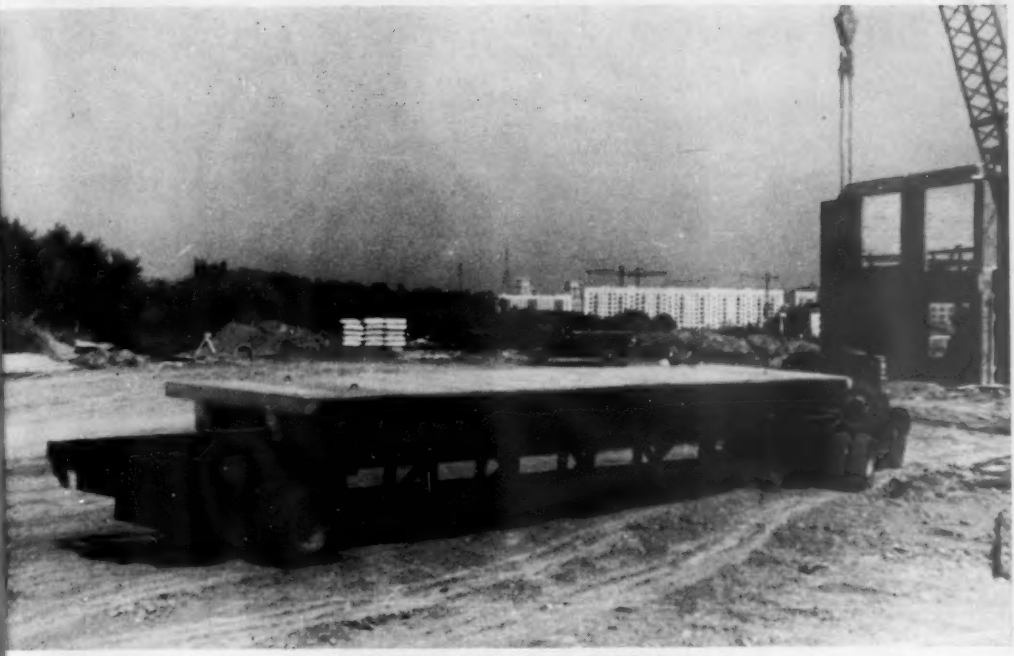


Fig. 20—Transporting tee's to the job site.

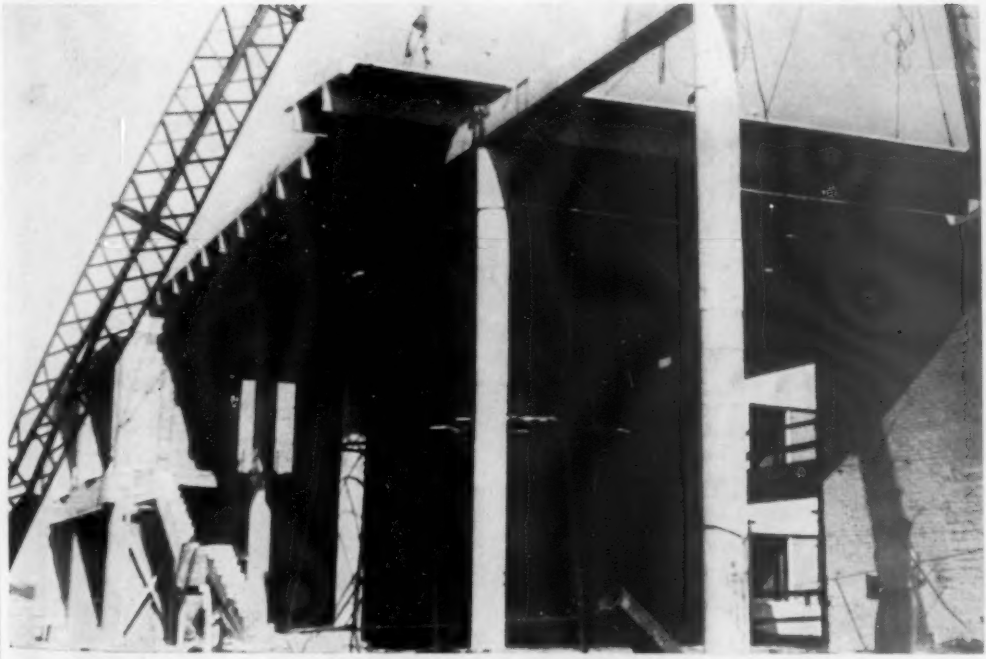


Fig. 21—Typical erection procedures.

Tall Precast Concrete Towers

by **Franz Sauter**

San Jose, Costa Rica, was selected as the site of the 15th World Amateur Baseball Games scheduled for April 1961. For this special event it was necessary to recondition the Antonio Escarre Baseball Stadium. This necessitated the construction of ten towers for night lighting with a required height of 100 feet above ground level.

The original idea of using steel poles of a tubular cross-section was discarded because of the high import cost. A conventional steel angle open-truss structure was planned with a base width of 9.5 feet, decreasing gradually to a 1.0 foot width at the top; but this design presented no outstanding characteristics.

Productos de Concreto Ltda. presented an alternative design in post-tensioned concrete, providing for the construction of the towers at the job site in a horizontal position and the final erection by means of a crane and proper equipment.

For the definite acceptance of this scheme various factors were taken into account. Economy: a comparative cost analysis showed a favorable difference for the prestressed towers which determined the choice of the type of structure. Time: the time available for the completion of the whole project, including the planning, was only 2½ months. With this time limit it would have been impossible to construct the towers as a steel structure, because of the difficulty in obtaining the required rolled angle sections locally, and because of the time involved in the preparation, welding and shop-assembling of the tower sections prior to their erection. Appearance: esthetic reasons were also a determining factor as the post-tensioned concrete design was especially trim and slender and would be an outstanding feature and symbol of the 15th World Amateur Baseball Games.

The towers are a rectangular hollow section; their width varies from 2 ft. 9 in. at the base to 1 foot at the top. The wall thickness of the box-section is 5 inches and remains constant through the whole length, except for a 13 foot section at the top, which is solid. The total length of the towers is 104.6 feet, rising 100 feet above ground level. Each one was prestressed with 8 post-tensioning tendons consisting of 12 wires 5.4 mm. (0.215 in.) in diameter. The LEOBA-prestressing method was employed with an initial prestressing force of 420 kips. Four of these cables are continuous throughout the length of the tower and the remaining four cables are anchored at about 56 feet from the base. A concrete strength of 5000 psi. was used.

The foundations are plain, rubble concrete footings 10.5 x 10.5 ft. with a variable depth of 6 to 10 ft. depending on the soil conditions. A square hole 3.5 ft. wide and 4.6 ft. deep was left with a lateral sloped ramp to facilitate the placing of the tower during its erection.

The embedded length of the tower in the foundation is only 4.6 ft. and was done by filling the free space in the hole with concrete once the tower was in the final vertical posi-

*Manager, Productos de Concreto Ltd. San José, Costa Rica.

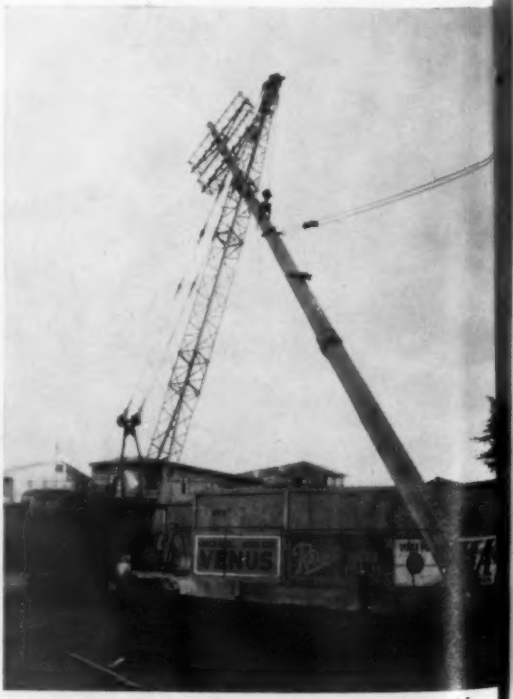
100 ft. prestressed light towers.

tion. High-early-strength-cement and a very dry mix with a low shrinkage factor was used for the concrete in the anchoring zone in order to obtain the required strength of 2500 psi in 14 hours. This enabled the removal of the guy cables and the erection equipment which held the tower in position during the hardening of the concrete. In this manner it was possible to quickly move the equipment to the next tower. The foundation is not reinforced except for a nominal amount of steel; large size boulders were used with the 1500 psi concrete.

The tower was picked up and erected at a point 29.5 ft. from the top. Suspended in this position the tower had to resist very unfavorable conditions which it will probably never have to resist again during all its life. It was necessary to admit temporarily higher stresses than permissible (tension 650 psi, compression 3100 psi) but the safety factor against rupture remained ample. In spite of suffering strong impact loads, such as the free fall from a height of two feet, the towers behaved in every sense as expected and demonstrated clearly the high resilience of prestressed concrete, i.e. the ability to resist large overloads and recuperate after removing them. The wind design load was an average of 25 lbs/sq. ft. of tower surface and 65 lbs. per reflector. Each tower was provided with a total of 32 reflectors.

Due to the lack of adequate equipment, the erection presented problems which fortunately were overcome by careful planning of all operations. A 10-ton crawler crane with a 75 ft. boom raised the towers to approximately a 70° position. The final erection was completed by pulling on a winch with a D-8 Caterpillar tractor; lateral stability was achieved by guy cables attached to smaller tractors. The total weight of each tower is 21 tons, and the reaction at the lifting point was 11 tons. A total of 10 towers were erected without accidents or faults of any kind at an average of one per day.

The design was done by the author. The supervision of construction and erection and all post-tensioning was carried out by Productos de Concreto Ltda., San Jose, Costa Rica. Labor and equipment were supplied by the Public Works Ministry, who took an active part in this project and who through enthusiastic acceptance of our design, made possible the realization of a project which is singular in its type in Central America.



Erecting tower with a small crane and a cable pulled by a dozer.

"The Philosophical Creative Approach"

by **Louis I. Kahn***

Presented at the Sixth Annual Convention of the
Prestressed Concrete Institute

I should like to talk about prestressed concrete, not as an engineer and one who even knows intimately the meaning, internal meaning, of concrete but as it impresses me and impresses many, many architects. What impresses me is the wonderful plasticity of concrete in general and the acceptable finish that concrete gives.

When I visited the work as it was going on in the Medical Center, the one thing that bothered me most of all was the presence of the crane that kept always humiliating the little things that were on the job that built the building. The columns always looked like matchsticks because the crane was so powerful in lifting the very heavy members, and they seemed as though they weighed nothing.

I resented it because I wasn't familiar with it. One day I had to wait some little while and I suddenly realized that the crane was a powerful instrument that was nothing but an extension of your arm. The crane is a design maker which

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Philadelphia, Pennsylvania

Medical Research Laboratories, University of Pennsylvania.

can actually motivate and create for you if you only but used it as your servant instead of your enemy. I began to visualize the wonderful things that can come from, right from the start, considering your design in terms of the crane.

I was not satisfied, then, with lifting only twenty-six tons or fifty tons, but I wanted to lift a hundred tons, 150 tons and even more. Through the encouragement of my engineers, Dr. Komendant and Keast & Hood, they told me this was all very possible.

What does it mean, architecturally? The architect reasons in terms of columns as a series of members which give him a musical rhythm. Columns which are a hundred feet apart are a different kind of rhythm, it's a boom-boom rhythm which isn't the delicate flowing rhythm of the Parthenon and other buildings of old.

I began to visualize what this column really is that's a hundred feet apart. It's an event, a strategic event, in the building. It is a point where many wonderful things can happen, and I thought in terms of this column as a room, not just a column, a heavy thing, but something which is a room, itself. I saw these series of rooms as the strategic point from which the beams and the expanding members come as stations. These stations represent the servants of the building or the servant of the spaces and, because their combing together was such an important event, the way you make it became of great importance.

If you thought in terms of concrete poured, one could certainly make wonderful forms, but there isn't the conclusive receptiveness in poured concrete. It is, by nature, a material which is accidental in its final appearance, whereas prestressed concrete has the advantage of a predictable appearance.

Now, the buildings which I visualize with these columns, which have become rooms, open up an entire new attitude in planning.

The columns as I saw them became the servant areas of the building, rooms serving the spaces of the interior. The joints which come together, the various elements that come together, I felt should be made of gold. I didn't mean, really, gold; I really meant that the joint became a very important aspect. It wasn't something that you homogenized into other members, but became a member in itself, so that when a member came together with another, something happened, that gave a higher decorated event to the column.

I saw in the beginning, because the architecture was getting bigger, that you can accomplish more with one stroke—then the stitching approach to architecture, little pieces, little pieces coming together with multiple joints. It may be a beautiful piece of cloth architecturally, but it doesn't have the potential of architectural concrete in it, and I felt that the larger the pieces the more I like it. Only the day before when I saw the crane, the pieces, I thought, must be small, manageable.

Now I said, "No, I demand more and more of the crane," and, because of this, I began to realize that the event of these large pieces coming together is the point from which wonderful decoration can occur. I believed that the joint is the beginning of ornament. It is not an applied thing; it's actually strategic, it stems from a strategic consideration of a building. It stems from a challenge against the elements. It stems from water tending

to destroy the building, sun tending to make the building uncomfortable, and those are the things that make joints expand and crack. These joints can be made to dramatize in the most wonderful way as you increase the size of the members coming to each other so that the joint becomes a high, decorative thing.

Even the roughness of the concrete becomes smaller in importance because, if you center around the making of this joint and think for a moment symbolically—that this joint is made of gold. It is the sculptor's chance for the expression of the meaning of this joint, in the same way the Gothic architect considered the gargoyle—which is only a way of carrying the water away from a roof. They began to see it as demons flying away from the church, that couldn't stand the glare of the glory of the church. Eventually all gargoyles were made in the form of demons, not facing the church but facing away from it, which came from the understanding of the water going down—I mean, always flowing down and finding its relentless position regardless of what you do, and the gargoyle was a way of carrying water away.

At first it was a very modest thing; then it became a highly-decorative thing because you understood it. These gargoyles were tremendous demons and they would spit the water and they became the high point of decoration or ornament, so likewise the making of this joint could be the beginning of a new architecture.

Prestressed concrete should try to consider the joint-making as not something which is a form of homogenizing one member with another, but making the event of the joint, a construction of itself. I proposed when I was in San Diego the other day, the idea of making the earthquake construction flexible, rather than rigid and, that it would be a marvellous opportunity to make a joint very visible, strongly visible. As a matter of fact, it need not be the exact size that is required because it is a precious event in the building; therefore, you can make allowance for lavishness and expression so that a man at a distance can recognize as in every work of art how it is made.

A good piece of cloth always shows how it is made; a good painting can account for every stroke; a piece of sculpture always shows the imprint of the finger or the imprint of the tool. These things are not thrown away, and so it must also be apparent when we make concrete or when we bring together great members at joints how they are made.

I can't speak enough about the beauty of it, the meaning of it and how it can be the emergence of an attitude toward the making of it, the emergence of a great architecture. The separation of members so that one never misses what each member does, is another part of the same family of thinking.

One little note that I have to add —that, the building of cities in which, architecture is concerned with, we have great changes coming about. We are still thinking in terms of cute little squares in Europe which we'd like to have in America. And the careful placement of one building with another, but we have relentless forces and we have democratic forces which are very different from the forces that made European cities.

We can make all the plans in the world and, the next day, circumstances will change them. There

must be a very strong order that must transcend all change and be powerful in its truthfulness so that change does not affect it. In fact, I believe that our democratic ways will eventually produce the greatest architecture because it will consider the greatest number of people as judges of that architecture.

The architecture of old was infinitely great and we depend tremendously on being inspired by what was done. While we are terribly discouraged at the chaotic conditions of architecture today, still I believe that the test, architecturally, will be a different test. It will be a test of a broad agreement with the artist which he can handle if he is big enough.

I think the artist will have to be a man who knows and feels many more things than the artist before. Many designers consider themselves artists today, but they are not, really. They are not big enough to be called artists. They can make things beautiful, but that's not the thing that's important. What is important is to set down those principles which are indestructable.

Now, there is a very important difference between the cities of old and the cities today, and that is that it stems from our way of moving and I speak about the architecture of movement, which is the architecture which the motor car and its wonder is bringing about.

I believe that in the center of town, for instance, a street wants to be a building—not just a street. It's no longer a pavement, but it is a building, because why should we have to rip a street every time we fix a pipe? A street is a place on which cars go and people go; this represents a great deal in the economy, and we should go downstairs and fix our pipe without bothering the street.

The street should be a building with one story where you fix pipes; one story below that you have some other event. Now, if you have the attitude that it's a shoddy construction below and with asphalt above, neatly laid as of yesterday and you're finished—well, that is a completely wrong attitude, especially in the center of town.

I believe there's a definite trend of architecture as distinct from the architecture, as I call it, of Picasso, which is an architecture of force. It brings people from distant places on complicated roadways and, when it comes to the center of town, it should become more delicate, more sensitive to the precious entrance and to the precious center of town. Thereafter, architecture must change in the center of town. You can't have the same brute construction with space below columns which is not used, which gather nothing but newspapers and discarded material. The whole building from the very bottom to the point where you rise must then be a building, in itself. Viaduct architecture is an architecture of force.

Why shouldn't this viaduct contain other piping? Why can't it contain the storage houses of the city; why can't it contain other necessary things like parking? The architecture of the viaduct becomes more important, more delicate, more sensitively attuned to the needs of the city. They are all strategic areas. They are the same as the great walls of a city which were equally important in olden times to the meaning of the city.

These viaducts that come in must become a different architecture as they get to the top. They can be poured-in-place concrete, let us say, outside but, when they come inside,

the most considered, respectable use of all materials must come into play because it is, to many people an inspiring source of knowledge. People learn about people in towns. Towns will never die because they are the point from which you learn about yourself, and anything you make in town should be of the complete best and with highly-considered ways.

I believe, as part of the design of the street should come the design of where you park. Parking should not be a real-estate affair, an opportunistic thing. It should be a part of the design of the street, the system of streets. I like to call these and other facilities gateways and I would like them to be placed so strategically that these large entrances or gateways are where they have to be, not just where real estate dictates, but where the system of movement dictates.

These gateways should be wonderful buildings. They should have a shopping center below. The core of it should be parking; outside should be a motel; on the fifth level should be a garden; over that should be the continuation of the motel or hotel or department store. It must be a strategic building, very close to the center and, because it is so expensive to be in the center, it must be other things, than just a garage.

A garage in the center of town is definitely an interruption of the progress of town; therefore, this viaduct architecture, as it enters town, must take this into consideration. And what material could you use but concrete? You couldn't use anything else! I say there is a wonderful opportunity for the architect and engineers to collaborate in the making of these structures as the real imagery of a new city.

It isn't the little sticks and stones —what I call sticks and stones. They are nothing compared to the architecture of movement, something which expresses the going and stopping. The architecture of stopping always is a beautiful architecture, but is is not the architecture of the garage, as we know it. It's the architecture of gateways, and the only material is concrete and the only way of making this concrete is in its highest perfection, in large parts, in great joints and in adding such consideration of parts that make the man on the street realize the greatness of engineering and the greatness of architecture.

From Architect's Conception To Concrete Reality

by **Thomas J. Leidigh***

Presented at the Sixth Annual Convention of the Prestressed Concrete Institute

The A. N. Richards Medical Research Laboratories of the University of Pennsylvania is a noteworthy building both in the field of Architecture and in the field of precast and prestressed concrete. The project includes three forty-seven feet square seven-story precast tower buildings, a poured-in-place service building, and eleven poured-in-place service shafts. This paper will deal primarily with the precast towers (including design, manufacture, and erection).

This building is the first, to the writer's knowledge, to use precast concrete for a building of this number of stories. It is also the Architect's first major project using precast concrete. Many of the concepts of structure integrated with mechanical services and Architectural expression are relatively new. The Architect's development of and belief in this integration is a topic in itself. This association makes it difficult to isolate the actual beginning of this building. The first recognizable encounter with the square tower concept was a tower, supported on four columns located at the midpoints of the sides of the square. The framing system also included two main girders crossing through the center of the square,

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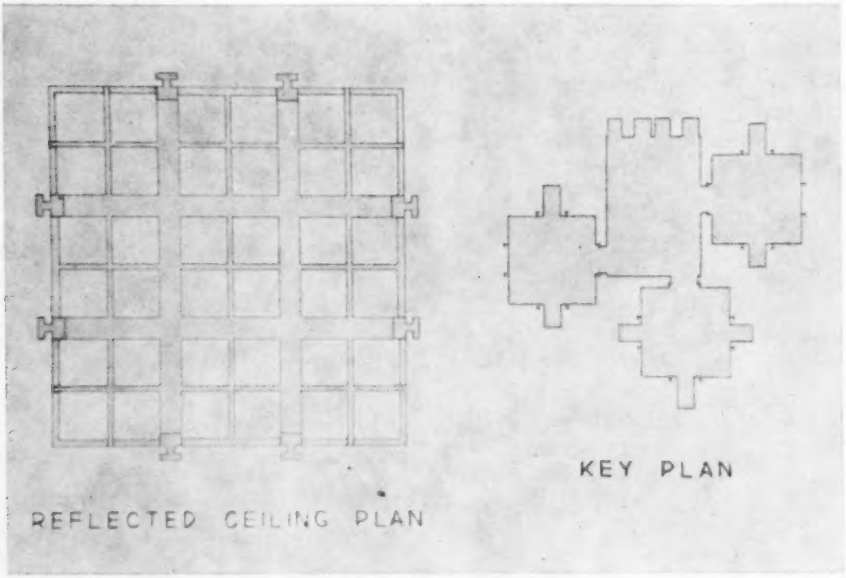

Fig. 1 Typical floor framing plan.

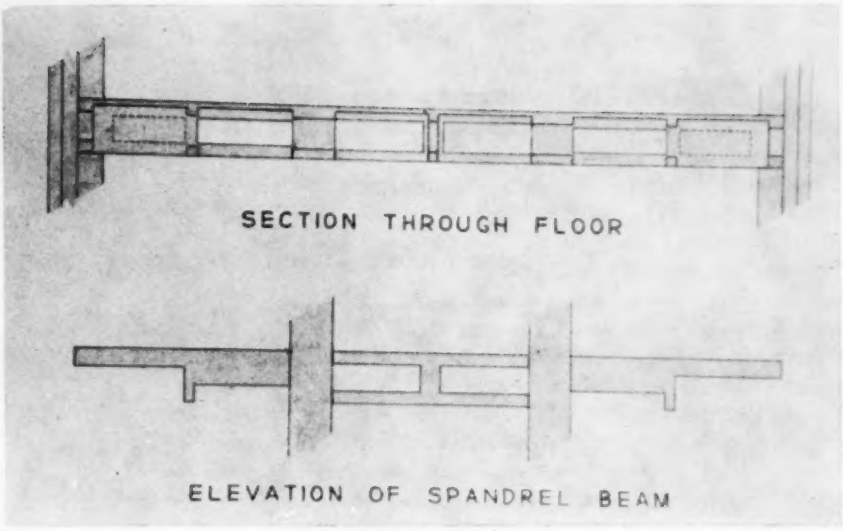

Fig. 2 Girder elevation.

one in each direction, and an open type floor system cantilevering from these girders. (Fig. 1)

In essence, this is the concept of the framing system. The single column at the center of each side actually became a pair of columns, located at the third points of the side; the main girder in each direction is actually a pair of girders crossing at the third points of the square, column to column; the open type floor system cantilevers from this system to the corners. This basic system, together with spandrel beams along each side, divides the square floor into nine smaller squares. These nine squares are subdivided into four smaller squares, each by secondary members at the midpoint in each direction. A poured-in-place slab completes the floor system.

The depth of constuction—top of slab to underside of structural members—is 3 ft. 6 in. All the interior members (main girders and secondary members) and the spandrel beams between the columns have a common bottom elevation. They are pierced horizontally by large openings to permit the passage of mechanical ducts, piping, etc., through the structure—that is, they are Vierendeel Girders. The bottom of the spandrel beams step up away from the columns, expressing the cantilever nature of the corners. (Fig 2) The openings in the spandrel beams between the columns permit the passage of mechanical services into the adjacent poured-in-place elements which join at these points. The size of all these horizontal openings vary with the ability of the member to withstand piercing at that point.

In brief form, the rather lengthy period of evolution and crystallization of this structural system was as follows: the establishment of the relative size of openings possible at various points in the structure; the preliminary layout of the mechanical services passing through the openings; the sizing of these services at the critical openings; the sizing of the opening required to permit passage of these services, with due

regard to pitch of drainage lines, etc.; and, finally, the sizing of the structural members necessary to accomodate these openings. Truly, this structure is more than just a structure—it is part of an integrated building mechanism.

The main girders are 34 in. wide, 38 in. deep and 45 ft. 7 in. long. There is a 4 in. additional poured-in-place slab above. The length is divided into six panels: a solid panel at each end (with an interior void to reduce weight) and a panel with a 6 ft. 4 in. long by 20 in. high opening next to it, and two panels between the intersecting girders, each with 6 ft. 0 in. long by 24 in. high openings. The bottom chords below these openings are, respectively, 10 in. and 8 in. depth. Top chords are 8 in. and 6 in., as cast, and 12 in. and 10 in. after addition of the 4 in. poured-in-place topping. Verticals at intersections with the girders are 2 ft. 10 in.—the width of the intersecting girders. In one direction, at each floor, these girders are monolithic units, pretensioned in both the top and bottom chords. In the other direction, the girders are in three sections (column to girder, girder to girder, and girder to column) post-tensioned in both the top and bottom chords.

The spandrel beams are 42 in. deep at the center, stepping up to only 14 in. deep at the ends. They are 46 ft. 5½ in. long overall. Bottom chords below these openings and top chords above these openings are 10 in. each in depth. The outside face of the spandrel beams is exposed, flush with the face of the wall.

The secondary members are 38 in. deep, plus 4 in. of poured-in-place topping. They are 8 in. in width, with 6 ft. 0 in. long by 24 in. in. high openings. Bottom chords below these openings are 8 in. in depth. Top chords are 6 in. deep, as cast, and 10 in. after addition of the 4 in. poured-in-place topping. In each of the 9 squares formed by the main girders and the spandrel beams, there is one secondary member, two panels in length; and two secondary members, one panel in length each.

The columns are of an unsymmetrical I shape, with 34 in. wide flanges, the outer one is 10 in. thick and the inner one is 20 in. thick. A short 12 in. thick web joins the flanges. Columns are in one-story pieces, 12 ft. floor to floor, with the top of the column flush at the underside of the floor construction. The column above is notched to fit over the end of the girder and the spandrel beam which rest on the inside flange of the column. (Fig. 3) The web and the outside flange of the column which extend beyond the face of the spandrel beam lap down on the outside to meet the column below, creating a continuous vertical element projecting beyond the face of the wall and interrupted only by the joints at each floor.

The main girders are designed as Vierendeel Girders. The top chord of the girders was prestressed in order to handle the high second-

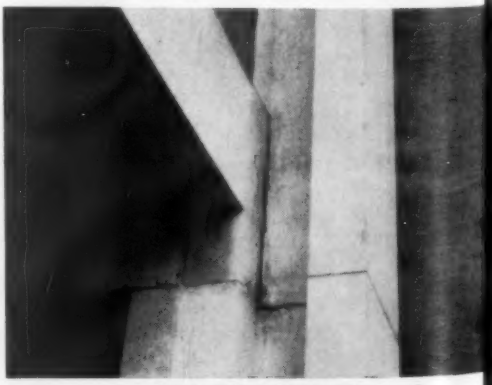

Fig. 3 Column connection.

Fig. 4

ary moments due to the shearing stresses, which were high enough to crack the concrete. The pretensioning in the bottom chord was, of course, greater in order to take care of both the main chord tension and the secondary bending moments. As previously mentioned, the girders in one direction at each floor were cast as one unit, and these sections were pretensioned. The girders in the other direction were cast as three pieces and were post-tensioned after erection. The direction of pretensioned and post-tensioned girders was determined by erection and post-tensioning requirements. The design of the girders in both directions is based upon an equal sharing of the total final load. The pretensioned girders were also investigated for their ability to carry the weight of the post-tensioned girders in addition to their own weight during the erection process. The poured-in-place slab above the girders is designed to act compositely with the girders. Shear keys and stirrups are provided to tie these sections together. The design included investigation before and after placing of this slab.

Notches and bearing seats were provided in all members so that erection required no temporary shoring of individual units. (Fig. 4 & 5)

The floor system is designed for a superimposed load of 100 pounds per square foot, made up of a 75 lb. live load and a 25 lb. partition allowance. The frame of the building is designed for a wind load of 25 lbs. per square foot. The majority of this wind load is carried by the side columns and the spandrel beams, rather than by the main girders, due to the greater relative stiffness of these members.

Concrete strength for the precast structure was 5000 psi; and for the poured-in-place slab, 3500 psi. Both high early strength cement and admixtures to increase the workability of the mixes were permitted by the specification but were not required, (however both were used). While the drawings showed certain strand sizes for the prestressing, the specification clearly permitted the use of alternate arrangements to secure the required prestressing forces. Substitutions could be made within the limitations included in the specification and with approval of the structural engineers.

The prestressed girders contained normal mild steel reinforcing in addition to the prestressing, and the other members were reinforced com-

Fig. 5

PRINT IN BINDING

pletely with mild steel reinforcing. All members were steam cured.

Members were cast in steel moulds, and the general surfaces were left as they came from the forms, without dressing. Edges were gone over lightly with a stone. Patching was kept to a minimum, and, where permitted, had to be practically invisible. Columns, which in some areas are exposed on all four sides, were cast vertically so that the only surface not in contact with the form was the top, which is covered by the girders and columns above. Girders were cast in the normal final position, with collapsible steel forms for the openings, and the only nonformed surface was the top, which is covered by the poured-in-place slab. Spandrel beams were cast on their sides, with the outside surface down. Secondary framing members were cast on their sides. Finish on the non-formed surfaces of the spandrel beams and secondary members was as near the finish of the formed surfaces as possible. In addition, the secondary members were erected so that the formed faces were always facing the same way within the limits of any one floor of a tower.

Erection of a floor began with the setting of the two pretensioned girders. (Columns were erected as the final step of the floor below.) The girders rest on the inside half of the inner flange of the columns and were set on steel shims. Normally a minimum of two shims were used with powdered graphite between them to facilitate sliding. Next, the six sections that make up the two post-tensioned girders were erected, again on steel shims. Shims on the columns were set to correct level by instrument. Shims on ledges of the precast girders were adjusted to bring the bottoms of the intersecting girders level. Tolerances in the precast members were premeasured in an attempt to minimize the trial and error aspects of this operation. Post-tensioned girders were stressed immediately following erection. The post-tensioning rods were threaded into place and bearing surfaces in line of the post-tensioning forces were packed with steel shim plates.

The post-tensioning procedure was as follows. The first step was placing the center bottom rod of both girders, then the two top rods of either girder, followed by the two outside bottom rods of that girder, the top rods of the second girder and then the bottom outside rods of that girder. The 1¼ in. bottom rods were stressed to 62 tons each, and 1 in. top rods to 35 tons each. The stress was measured by gauges on the pumps activating the jacks and was checked by elongation of the rods. Erection of some of the secondary members and the two spandrel beams which did not foul the ends of the post-tensioned girders proceeded during the post-tensioning operation. Spandrel beams rest on the outer half of the inner flange of the columns and run past the ends of the girders. Grouting of the rods followed the tensioning and was completed before erection of the remaining spandrel beams. Packing and pointing of joints followed, but did not delay succeeding work.

Columns were erected after the spandrel beams were in place. They were set on steel shims, again a minimum of two shims with powdered graphite between, to facilitate jockeying into final alignment. Succeeding columns were fastened to those below by 9-1¼ in. round mild steel rods, threaded through sleeves the length of the columns and into sleeve nuts at the top of

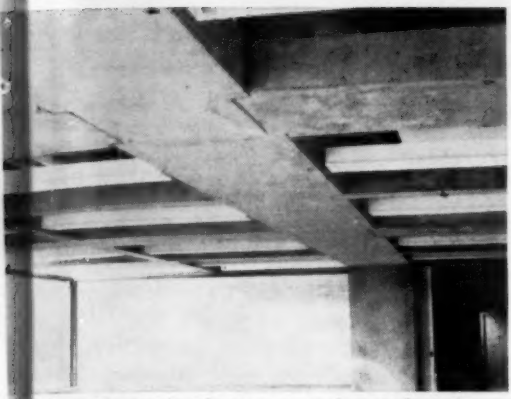

Fig. 6 Precise fit of precast members indicated by this completed framing photo.

the column or beam below. Following final alignment of columns, joints below were packed and then the rods were tensioned to 20,000 psi using a torque wrench to seat sleeve nuts, which were then ready to take rods from above. Columns thus cantilevered to the floor above, and when movement of the girders above took place during post-tensioning, the girders slid on the ends of the cantilevered columns without disturbing the alignment of the columns.

Erection of the three seven-story towers took place in less than three months' time. The speed at the start was quite slow, but as everyone learned his job the speed picked up, and at the peak of operations, the rate was one floor on each of the three towers in a week's time. (Figures 6 & 7)

Concreting of the poured-in-place slabs followed the erection closely. Forms were supported on inserts cast into the precast members. The normal condition, as erection proceeded from tower to tower, was that the slab on a tower was poured and ready as a working deck by the time the erection returned to that tower.

Two load tests were conducted on the structure. A single pretensioned girder was test loaded in the casting yard by jacking against a test frame. Load for this test was approximately equal to live load. This load was limited to keep the secondary bending stresses in the top chord, within limits which would not crack the concrete section. This was done because the additional 4 in. of poured-in-place topping had not been placed. The deflection under this loading was just under ¼ in. A completed floor was test loaded following erection and pouring of the 4 in. slab. The load was applied by jacking against the floor above, with shores in place to two additional floors above in order to distribute the load. Four jacks were used, placed at the intersections of the girders. Total load applied was 150 tons (52,500 pounds per jack). This load produced shears and moments equivalent to twice the live load plus the partition load which was not yet on the structure. Deflection under this load was about 1/12th of an inch. Deflection calculations for the composite structure indicate this deflection is consistent with that of a girder fixed at the ends by the columns. This is in conformance with the design assumptions.

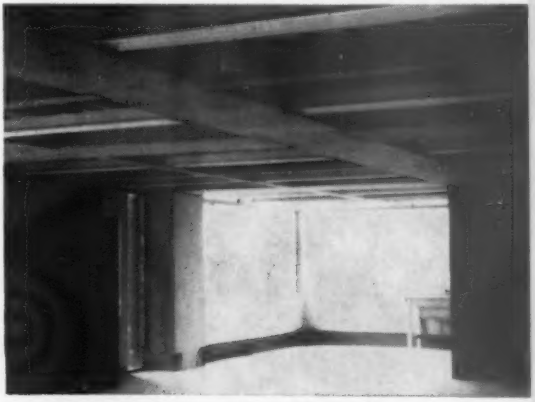

Fig. 7 Photo of a column-free cantilevered corner.

VOLUME TIGHTLY BOUND

PROCEEDINGS PAPER

"CONCRETE RESULTS"

by J. B. Smythe*

Presented at the Sixth Annual Convention of the Prestressed Concrete Institute

The structural framing of the University of Pennsylvania Medical Research Building has been vividly described in numerous articles and papers since its construction. The careful thought and planning which went into its design are obvious. The problems of construction which had to be overcome, in retrospect, seem straight forward and not unduly complicated.

At the time our company first started to translate the contract plans into shop drawings for the individual pieces, the picture was not quite so sharply defined in our minds.

There were a few things which stood out clearly in the requirements for the work, and these provided the basic elements for making all other manufacturing decisions.

First and foremost was the fact that towers had to be built to a seven story height using precast concrete members. Also when all dead loads were in place and when all prestressing forces had been applied, the framing had to be square and true and accurately positioned, with respect to a cast-in-place reinforced concrete tower which formed the central core of the building (Fig. 1)

All those with experience in manufacturing prestressed concrete are aware that changes in length due to creep, shrinkage, and plastic flow take place when heavy prestressing forces are applied to the concrete; and that these changes of length are proportionate to the properties of the concrete at the time the forces were applied.

A glance at the plans showed that half of the main floor girders were of the pretensioned variety and the other half were sectional post-tensioned members.

It was apparent then that one half of the main members (the pretensioned ones) would be prestressed when the age of their concrete was approximately 48 hours. The other half of the main members would be prestressed perhaps two or three months after their manufacture.

The main floor members sat atop the columns on one half the thickness of the inside flange of the "I" shaped columns. The spandrel beams rested on the other half of

Fig. 1 Erection shot of the Medical Research Building.

*Atlantic Prestressed Concrete Div. of Warner Co.

the inside flange thickness. The column prestressing passed through both. Therefore, any variation in the length of the main members could affect the plumb of the columns or vice-versa.

While in most instances, a total tolerance of ⅜ in. was available for adjustment at these joints, the position of the spandrel beams on the columns had very little adjustment because the four spandrel corners absolutely had to meet properly to make the corners of the towers. Variations in these corners would have been very noticeable in the finished structure.

The other line which we felt would be most noticeable in the finished tower was the plumb and alignment of the columns. The columns form a principal feature of the exterior architectural treatment, and any variation, apparent to the eye, would be objectionable.

Thus we concluded that both columns and spandrel beams had to be erected to the best possible alignment without regard for the other members. Yet, the other members had to interlock properly with columns and spandrels and these were the members subject to application of heavy prestressing forces. Thus, their change of length had to be anticipated and provided for.

The next consideration from a manufacturing point of view was the manner in which all members would be formed. As noted above, a main consideration was that the manufactured pieces be of exactly the proper length. It was our feeling before manufacture that all tolerances provided in the design had to be preserved for the erector, and, therefore, that the pieces as manufactured had to be nearly perfect dimension wise. (Fig. 2)

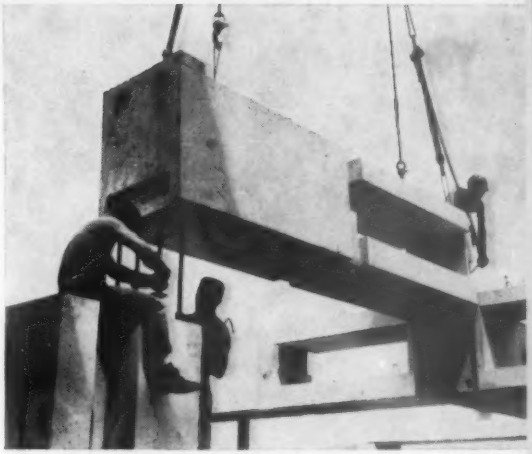

Fig. 2 Fabrication exactness required is indicated in this photo.

The first criterion for forming schemes was the production of true shapes. A second criterion, and equally important, was the selection of which surfaces would have formed finishes and which would have trowelled finishes. Architectural considerations required that all exterior surfaces of the building have formed finishes. On the interior of the building, it was deemed that the underside of the members were the most important from an appearance standpoint.

The exterior face of the spandrel beams was smooth and free of detail while the inside face had seats for the secondary framing, ledges and dowels for the floor slab, and some intricate projections at the corners. This inside face, of course, was the logical face to form, but since the outside had to have a formed face as well, we considered pouring the spandrels in an upright position. The quantity of reinforcing and thickness of the member made the risk of honey comb too great in casting with upright forms. Thus, the spandrels were poured lying on their side.

The spandrel forms were detailed with a flat pallet for the outside face, in which were cut the holes for the truss openings. All side forms

were to be wedged against the bottom pallet. The forms for all details on the interior face were supported by frames spanning and bracing the sideforms.

The pretensioning of the main members dictated that they would be cast in an upright position. This involved the use of top chord forms which had to be adequate to support a substantial weight of wet concrete and yet which would strip quickly and easily.

A final feature considered with the formwork was the type of form finish desired. It had to be dense, free of water bubbles and of as light a color as could be produced using ordinary cements, while maintaining uniformity of color in all pieces. White cement was not specified and not used anywhere in the precast construction of this building.

All considerations seemed to point directly to steam curing. It would help manufacturing schedules by accelerating the curing cycle while producing a whiter concrete in appearance. It would minimize the problems of creep and shrinkage. It would also ruin wooden formwork.

Contrary to the opinions of many people, it has been our experience that carefully made wooden forms constructed using plastic coatings and facings in some instances can produce results in every way as pleasing as can be obtained with concrete or steel forms.

When all things were considered, however, the use of steam curing, 23 floors of repetitive members, the accuracy of shape and dimension required, we chose to fabricate all the formwork out of steel.

We did impose a few rules with an idea of achieving maximum utility from this substantial investment. The first was that spandrel sideforms were to be of such a size and shape as to be useable for the forming of rectangular columns and small members in succeeding plant work. The second was that sideforms for main floor members were to be designed for rehabilitation into standard universal bed sideforms such as are used for building bridge girders. Thirdly, all secondary forming of pockets, seats, recesses, etc. was to be done in such a manner as to minimize the wear and tear on the pieces we hoped to make a part of our regular plant equipment.

On the other side of the ledger, we decided to use somewhat lighter liner plate for the bulk of the forms than we would buy for regular plant use.

The one remaining formwork item not yet discussed is the column form. The same criteria of steam curing etc., applied and steel was selected as the form material. The columns were different than the other pieces in that they stand vertically and from an appearance standpoint should be cast vertically. Acceptable alternatives were not to be found and form details were investigated.

The columns as shown on the original architectural drawings were of an "I" shape with all flange surfaces parallel. Therefore, unless the boxes making the web of the "I" were collapsible, it would be impossible to remove them. On self supporting forms of this size which could be handled only with a crane, we did not want any collapsible sections to reposition exactly on each succeeding pour.

A conference with the architect and engineer resulted in permission to provide a taper on the inside face of the outside flange. This provided the means whereby rigid boxes could be used to form the web portion of the "I" shape.

The side forms were designed using steel plates stiffened with angles. The four sides were made with overlapping corners which bolted together to make a rigid box. Ties across the top and hollow cylindrical stubs on the base positioned the flexible tubing which was provided to make the openings required for the post-tensioning rods. These flexible tubes were internally stiffened with steel pipe and the mild steel reinforcing cages were tied to the tubing.

In ordering the forms, consideration was given to the delivery schedule. It was noted that the critical items would be spandrel beams and the secondary trusses. The secondary trusses required attention because of the large number required. We felt the spandrel beams would be a problem because of their complexity, both in shape and reinforcing. Our answer was to provide additional soffit liners for these items so that, by using removable sideforms, we could maintain a daily pouring cycle on both items.

As we had elected to provide forms for an entire universal bed, we would be able to set up and pour either six of the pretensioned main trusses or eighteen of the post-tensioned main trusses at one time. About half of the forms were fabricated by a company specializing in that work. The other half were fabricated by a local shop which regularly does steel fabrication for us.

The organization for the actual production was in accordance with our usual practice on all of our work. The work was divided into prestressed items to be produced on our universal beds and precast items having conventional reinforcing.

A work force of fourteen men, a welder and a foreman was utilized in the precast area. This force was able to produce two spandrel beams and four secondary trusses each day. Their work included positioning of all reinforcing in the forms, fastening all inserts, sleeves, and miscellaneous items cast into the members. They were responsible for pouring the concrete and doing the finishing as well.

A major item of work was the fabrication of reinforcing cages. This was handled by a separate force of eight men and foreman who produced cages for two spandrel beams, four secondary trusses, and two columns, each working day. All gangs were sub-divided according to the different pieces to be produced.

The cages for the universal bed work were produced by the work force in that area which did the pouring as well. The problem of the cages was somewhat different in this area because the pretensioning strand in half of the main members precluded the prefabrication of cages. All steel was tied to these pretensioned strands after tensioning, and was included in the work of setting up the beds for pouring.

As soon as production had started, it was apparent that the mild steel reinforcing in all the members presented problems not found in ordinary work. The problem was in the bending tolerances required to fit the cages together. Our reinforcing fabricator was able to produce bars bent to ¼ in. tolerances, and this proved adequate. In fact, most bars furnished were practically perfect. The fabricator used jigs in his plant to check the pieces before shipment and we used jigs in fabricating the cages so that very few problems were encountered once the requirements were agreed upon.

A few unusual manufacturing practices might be of interest to note.

In both precast and prestressed areas, a practice of double steaming

was instituted. Each piece was steamed after pouring to provide proper curing as noted earlier. After stripping the forms, the piece was examined for defects. Any patching required was done immediately and the piece steamed again. This second steaming blended the patches so as to make them invisible in most cases. It also whitened the color somewhat further.

By producing the columns in an upright position, it was necessary to turn them on their sides for storage because of the cutout in the base. This had to be accomplished without chipping the sharp square corners of all edges. Turning was accomplished by lowering the column gently into a pile of white sand. To my knowledge, no chipping of edges occurred in this handling.

When production was about 60% completed, we agreed to begin the erection work. We engaged a subcontractor for this work. As this appeared the most critical part of the operation, we departed from our usual procedure of taking bids.

We elected to handle the field concrete work ourselves. This consisted of drypacking and pointing of joints and grouting of the ducts in which the post-tensioned bars were encased. To accomplish this, we provided a superintendent on the job who arranged with the general contractor for the use of his masons as required for the concrete work.

This superintendent maintained liaison between architect, engineer, general contractor, erector and ourselves during the entire project. In addition to this superintendent, every post-tensioning operation was supervised by a member of our engineering staff working in conjunction with the supervision supplied by the designing engineers.

There is nothing I can say which would give adequate praise to the work of the erection crew. They solved every problem which came up during the work and by the end of the job were erecting at approximately 1½ times the best speed we thought possible in the beginning.

The site of the work was such as to provoke some sense of frustration. The towers were bounded on both sides by existing buildings which prevented passage of the crane from one side of the building to the other. A number of trees surrounded the site and protecting them caused some concern in the maneuvering of large trucks to the proper unloading positions.

In general, the transportation problem was a major headache from start to finish. Access to the site was provided via a footpath and exit was via a narrow driveway. One way traffic with little room to maneuver was the result.

The highway routes to the site involved the traffic of downtown Philadelphia, which continually scrambled arrival times. A limited amount of storage space at the site was available but for the bulk of the work, erection directly from the trucks was mandatory.

Two engineering considerations affected erection sequences. One was the requirement that the casting of the floor slabs was to lag behind the erection of the precast members by not more than one floor. The other was the fact that grouting of post-tensioned bars had to be performed in sequence with the other work.

The sequence finally determined as best was based on working two towers at a time. One tower was erected until the post-tension bar grouting sequence was reached. Then erection of the second tower

proceeded during this interval. As soon as possible, the erection of the floor of the first tower was completed and work began on the third.

The third and second towers were worked on one at a time with work transferring from one to the other to accomplish the grouting without holding up the work on either.

Once this procedure was instituted the work proceeded smoothly with the general contractor being able to pour his floors without interference in whatever tower was not being erected that day.

A number of interesting field expedients were utilized by the erector in executing his work. The most unusual was a sixty foot footbridge complete with handrails which was suspended from a crane for the operation of installing the post-tensioning bars into the main framing.

The length of these bars incidentally dictated the choice of direction of the main framing on some floors because of the proximity of the adjacent buildings. It was possible to interchange the directions of pre-tensioned and post-tensioned pieces at any level and this was done on the two top levels to minimize the reach required to set the main members there.

Another interesting device was the column plumbing jig. This consisted of a steel frame clamped to the column to which could be attached either a jack post or a come-along. With this jig, it was possible to rotate, translate or tilt the column a small amount. Two men carefully aligned each column while the main erection work was going on elsewhere and this system contributed to the excellent control of dimensions maintained throughout the erection work.

I was told by a representative of the general contractor after completion of the job that their checks indicated every piece measured was within ⅜ in of its theoretical position, and that all pieces on adjacent floors were within ¼ in. of perfect relative alignment.

My own spot check measurements revealed many locations where the dimensions between the cast-in-place tower and precast towers were within 1/16 in. of plan dimension even though both towers were laid out independently. This testifies to the fine control maintained on the cast-in-place work as well.

The window installation proved out the overall control achieved, because the windows were all of standard size. At the corners where the spandrel beams joined, mitered glass was installed without difficulty.

Looking back on the job, it seems now as though it wasn't as difficult as it looked in the beginning. However, the thing that really stands out in retrospect was the degree of cooperation exhibited by everyone connected with the project. We received every possible assistance from the architect, the engineer and the general contractor.

From this project, we learned not to fear the complex and difficult structure. Invariably, it is better planned and more carefully thought out than simpler work. We welcome assignments such as the University of Pennsylvania Medical Research Building.

A Sliding Joint, Prestressed Concrete Pavement

by Lyle E. Young*

Introduction

One important factor in the design of a concrete pavement is the necessity of joints to accommodate dimensional changes due to moisture content, temperature changes, or external loads on the pavement. It has been the conventional practice to provide joints placed perpendicular to the longitudinal axis of the pavement at suitable intervals. Dimensional changes are accommodated by the opening or closing of these joints. A bituminous or rubber material is placed in the joint to seal it against the entrance of water or foreign particles. At high temperatures the joints close, forcing much of the sealer out of the joint. With subsequent cooling, there is no positive action to return the sealer to the joint as it opens. A joint that is not sealed will allow passage of water or accumulations of foreign material which will impair the function of the joint or cause subgrade pumping with subsequent random cracking or deterioration of the slab.

To prevent pavement cracking prestressing has been employed. If prestressed wires are used to place the concrete in initial compression, subsequent stresses due to external loads or dimensional changes that would normally cause tension and cracking can be cancelled out. The placement of prestress reinforcing in a longitudinal direction, however, involves construction problems necessitating an interruption in the continuous process of placing concrete.

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This paper presents a modification and extension of a paving system similar to that employed by Freyssinet[1,2] in the experimental runways for Orly Aerodrome in Paris and the London Airport. The system proposed herein eliminates joints which open and close and provides for prestressing in a continuous paving operation. Any solution to a problem as complex as continuous paving is likely to introduce some new problems. It is hoped that this system solves more problems than it creates.

Description of Paving System

Figure 1 is a drawing of a precast concrete slab in the form of an equilateral triangular prism. One edge of the slab has a tongue as shown in Detail A, a second edge of the slab has a groove as shown in Detail B and the third edge has provisions for anchoring post-tensioned reinforcing strands, Detail C. Oversize holes are provided about halfway between the top and the bottom of the slab and in directions parallel to the tongue and groove edges. Reinforcing in top and bottom of slabs in a direction bisecting the 60° corner angles would probably be necessary to prevent the corners from breaking off.

Figure 2a is a plan view of the triangular slabs laid in place to form a single traffic lane. A thin layer of a plastic material is used to seal the joints, AB, BC, CD, etc., and to allow a sliding action. Strands of reinforcing are strung through the oversize holes, stretched within their elastic limit, and anchored at their

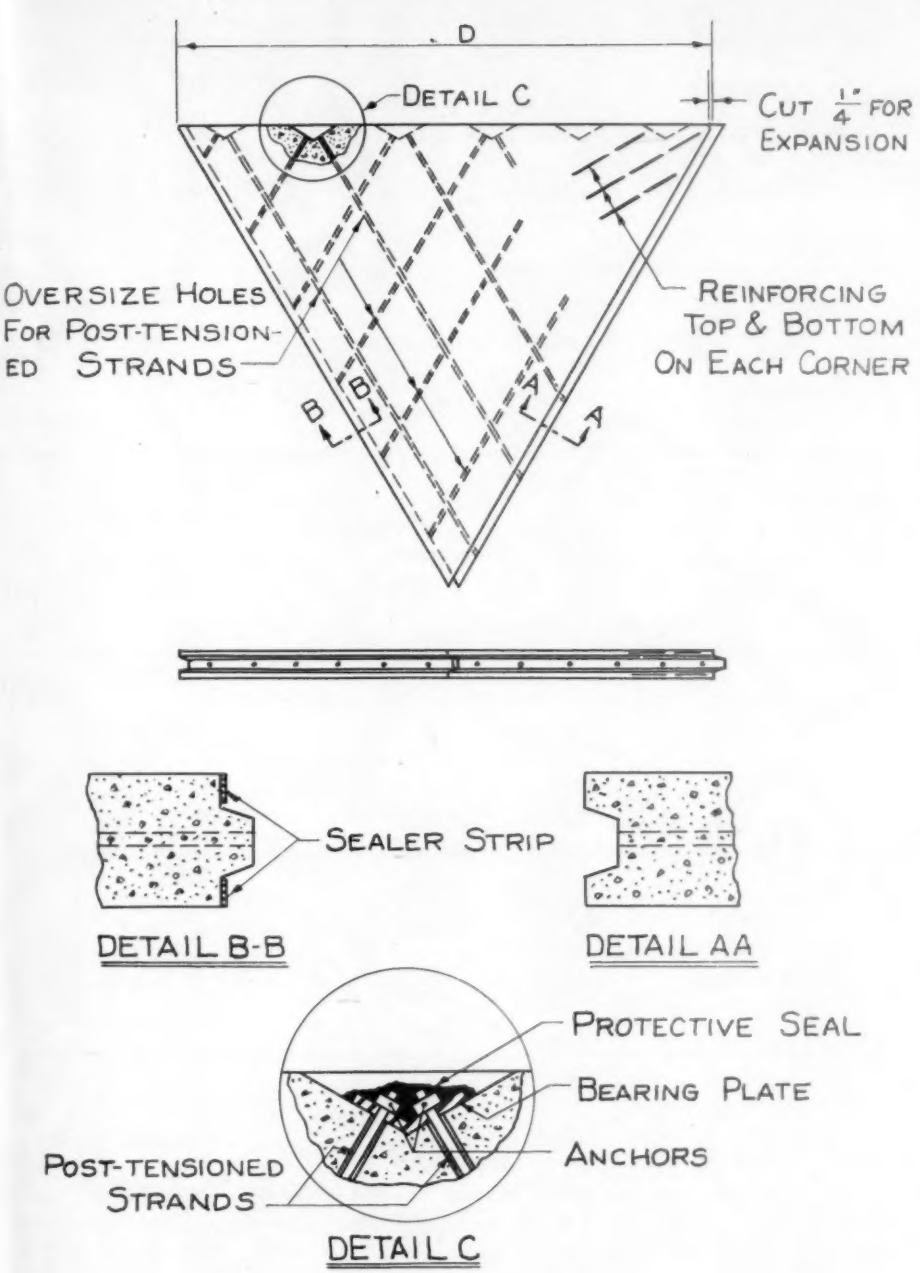
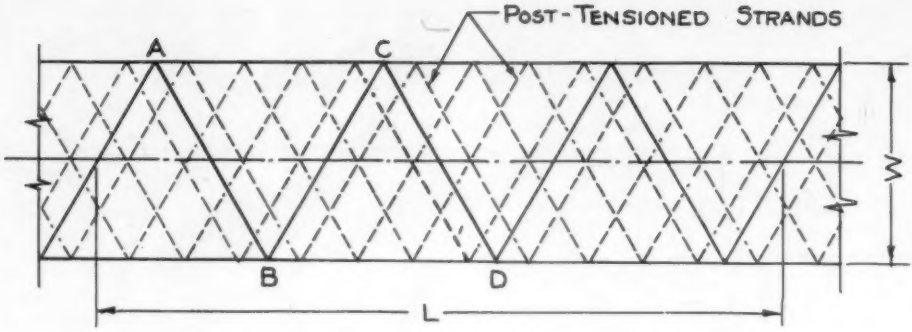
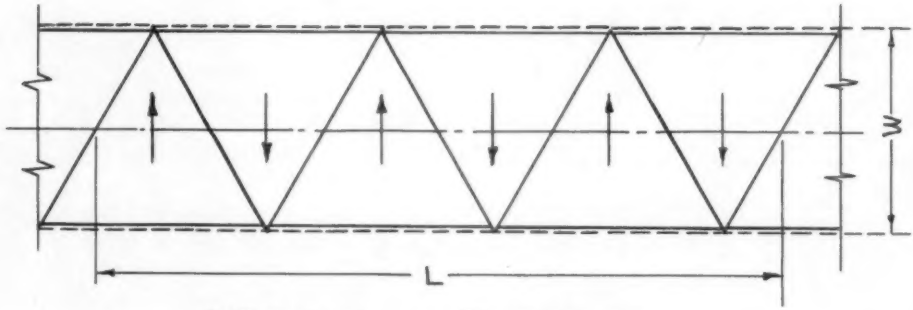


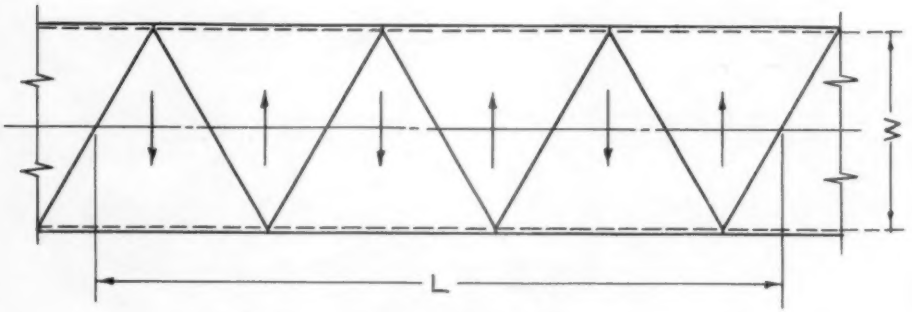
Fig. 1—Precast Triangular Slab Units.



a. MEDIUM TEMPERATURE



b. REDUCTION IN TEMPERATURE



c. RISE IN TEMPERATURE

Fig. 2—Plan View of Pavement Showing the Effect of Temperature Changes.

ends along the edges of the pavement. Figure 2b shows the relative movement of the slab (exaggerated). As a reduction in temperature occurs, the joints are kept closed by the elastic action of the post-tensioned strands. Sliding of one slab with respect to adjacent slabs occurs along the joints. Figure 2c shows the relative movement (exaggerated) of the slabs as a rise in temperature takes place. The movement of a slab is caused by the lateral component of the pressure of adjacent

slabs seeking space for their expansion. This movement also causes some increase in the tension in the strands which is available to keep the joints closed as subsequent cooling occurs. In Figures 2a, b, c, the length of a given portion of paving remains constant but the width, W, of the paving varies as shown.

The plastic material to seal the joint and allow slipping could be a semiliquid sealer lubricant or it could be a thickness of a solid material similar to rubber that would allow differential motion by shear slippage. Figure 3a shows a short length of two adjacent slabs with a layer of a rubber-like material between them. As slippage occurs, Figure 3b, the seal deforms by shear. The total differential movement of adjacent slabs for a 200°F change in temperature and a 12 foot wide pavement would be less than ¼ inch. Load transfer and vertical alignment of adjacent slabs is taken care of by the tongue and groove joints.

The forms for the precast slab units may be designed so that distance D, Figure 1, can be adjusted for different degrees of highway curvature. Figure 4 shows slabs placed for a horizontal curve. The outside edges of the slab units would have D increased or the inside edges would have D decreased or both. Vertical changes in slab placement for vertical curves or superelevation are so gradual on modern highways that the use of the plane surfaces of individual units to approximate a continuous curved surface should not result in a significant loss of riding quality.

Construction Procedure

The triangular slabs must be made with accuracy and care. This could be done at a central plant using precise steel forms and efficient assembly line manufacturing techniques. Under these conditions a more rigid control of aggregate, cement, water proportions, and curing could be obtained. The manufacture of slabs could be continued on a year around basis so as to enjoy the benefits and economy of a skilled and stable labor force. The application of the sealing strips could also be carried out at the central plant.

Suitable preparation of subgrade for precast slabs might appear to be a formidable problem. There has been considerable support to the idea that flatwork must be "cast in place" if it is to be uniformly supported. However, actual examination of cast-in-place flatwork subgrades has shown that in a short

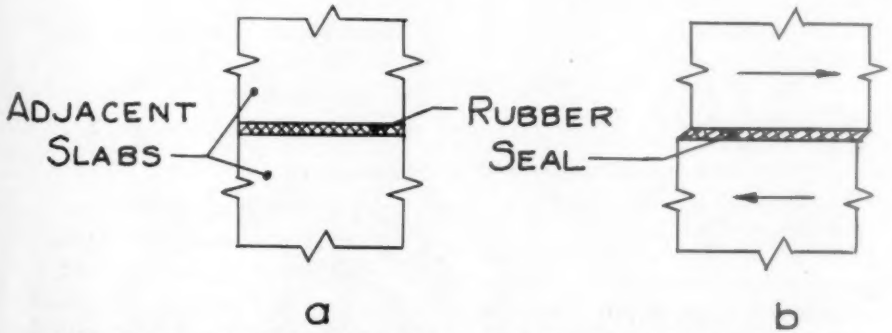


Fig. 3—Plan View of a Short Length of Adjacent Slabs Showing Deformation of Seal Due to Sliding Action.

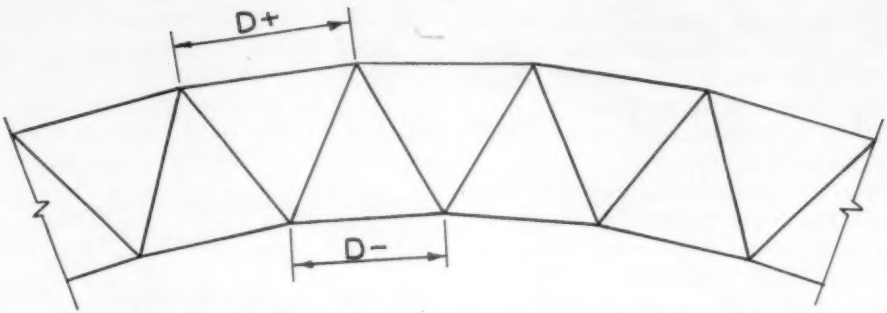


Fig. 4—Plan View of Horizontal Curve Showing Adjustment in Slab Dimension "D".

erosion, or plastic flow of the subgrade. It would seem that reasonable care in subgrade preparation might result in as good a support for precast slabs as is obtained for cast-in-place slabs. The use of post-tensioning to minimize tensile stresses in the concrete would make the triangular slabs less vulnerable to random cracking due to nonuniform support. Subgrade preparation would consist of laying a sand cushion with a suitable alignment and grade.

Slabs could be laid under a greater variety of climatic conditions than conventional construction allows. The slabs could be unloaded and placed by a crane working from the shoulder or ditch or from the paving already laid. The holes for the reinforcing could be used as a basis for designing a sling for handling the slabs.

Bars, wires, or cables of a high strength steel with a head or anchor preattached to one end would be strung through the holes. Post-tensioning could be done with hydraulic jacks. The amount of tensioning would be determined by the construction temperature. The post-tensioning of the strands and the maintenance of tension in the strands at any location depends upon longitudinal restraint of adjacent pavement. If there is no longitudinal restraint the post-tensioning forces will be dissipated by unlimited lateral movement of the slabs. In a continuous system, longitudinal restraint is provided by the subgrade frictional resistance of a sufficient length of adjacent pavement. At the terminals of the paving, abutments would be required to provide the necessary resistance to longitudinal movement. During construction the post-tensioning would have to be applied in steps and at some distance behind the placing of the slabs on the subgrade so as to make use of their subgrade friction to restrain longitudinal movement. Slabs would have to be placed similar to the position shown in Figure 2c so that the dimensional changes in the slab and joint due to post-tensioning would result in a positioning of slabs as shown in Figure 2a. The exposed ends of the strands and anchors would have to be protected from corrosion.

After the field procedure is complete, the pavement would be ready for immediate use by construction vehicles or traffic.

Maintenance

Joints which do not open and close but rather slide will minimize the problem of sealer ridges on the surface, or the passage of water or foreign material through or into the joint. A smoother riding joint with

smaller wheel impact loads will result. Joint maintenance for sliding joints should be considerably less than for conventional joints.

The provision of frequent joints to relieve stresses due to temperature and moisture changes might allow a reduction in slab thickness. The use of post-tensioned strands to prevent tension cracks in the concrete should almost eliminate random cracking.

Slab units could be taken up, moved and relayed for highway relocation.

A longitudinal joint between traffic lanes of a somewhat conventional design might be used. However, this problem might be reduced by combining two triangular units into one diamond shaped unit spaced alternately with regular triangular units as shown in Figure 5. All dimensional changes would be taken up by the lateral movement of the triangular slabs.

Summary

A comparison of construction and maintenance costs between conventional paving and the sliding joint system proposed in this paper is very difficult. The cost of precast slabs, subgrade preparation, placing and post-tensioning is dependent upon the development of efficient manufacturing and field techniques. The profitable investment of time and money in developing efficient techniques is a function of the quantity to be produced. Evaluation of maintenance costs are very difficult without long field research studies.

It is hoped that the ideas presented in this paper will stimulate other ideas and lead to a better way of surfacing highways.

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2. Netter, M., *The Prestressed Concrete Runway at Orly*. Annales de L'Institut Technique du Batiment et des Trauaux Publics (Paris) No. 5, Jan., 1948

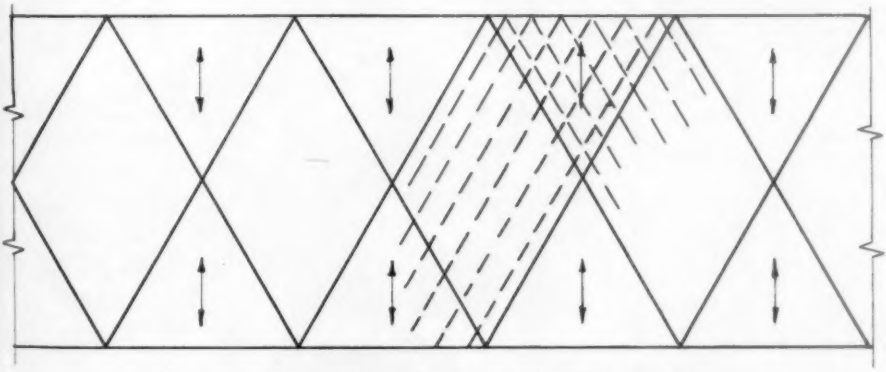

Fig. 5—Use of Alternate Diamond and Triangular Slabs for Two Lanes.

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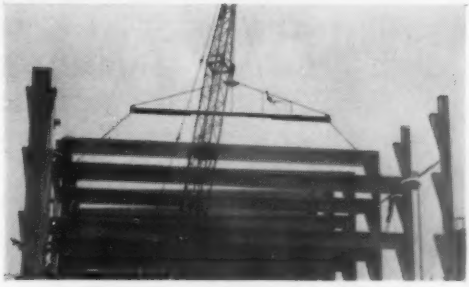

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Readers' Comment

DISCUSSION

"THE ALLOWABLE F_o-e REGION FOR PRESTRESSED SECTIONS AND AN EXACT METHOD FOR OPTIMUM PRESTRESS FORCE MODIFICATION"*

by **Ti Huang**

Associate Professor, New Mexico State University

The authors are to be commended for their work in the development of the equations of curves bounding the allowable F_o-e region for prestressed composite sections, and the method for prestress force modification.

The curves α, β, γ, and Δ are actually represented by the limiting equations (15), (16), (17), and (18). The combination of equation (15) with (11), (16) with (12), (17) with (13) and (18) with (14) results in the following system:

$$-\frac{F_o}{A_c}-\frac{F_o e}{Z_{bb}}+f^b_{D+L}\leqq f^b_{tp} \qquad (15')$$

$$-\frac{F_o}{A_c}+\frac{F_o e}{Z_{tb}}\leqq f^t_{tp} \qquad (16')$$

$$-\frac{F_o}{A_c}-\frac{F_o e}{Z_{bb}}\geqq f_{cp} \qquad (17')$$

$$-\frac{F_o}{A_c}+\frac{F_o e}{Z_{tb}}-f^t_{D+L}\geqq f_{cp} \qquad (18')$$

Using the equal signs in the above system, the resulting equations represent the curves β, α, Δ, and γ in that order.

Plotted wth F_o and e as the two coordinating variables, all four curves are rectangular hyperbolas. However, as pointed out by Magnel**, these equations are all linear in terms of $\frac{1}{F_o}$ and e. Equations (15')

*March, 1961, Vol. 6 No. 1, by K. K. Kienow and C. A. Bryan.

**G. Magnel: "Prestressed Concrete", McGraw-Hill Book Company, Inc.; New York, 1954.

through (18′) can be rearranged into the following form:

$$e \geqq Z_{bb}(f^b_{D+L} - f^b_{tp})\left(\frac{1}{F_o}\right) - \frac{Z_{bb}}{A_c} \qquad (\beta)$$

$$e \leqq Z_{tb}f^t_{tp}\left(\frac{1}{F_o}\right) + \frac{Z_{tb}}{A_c} \qquad (\alpha)$$

$$e \leqq -Z_{bb}f_{cp}\left(\frac{1}{F_o}\right) - \frac{Z_{bb}}{A_c} \qquad (\Delta)$$

$$e \geqq Z_{tb}(f^t_{D+L} + f_{cp})\left(\frac{1}{F_o}\right) + \frac{Z_{tb}}{A_c} \qquad (\gamma)$$

Each one of these equations represents a straight line. The e-intercepts of these lines are $-\frac{Z_{bb}}{A_c}$ and $\frac{Z_{tb}}{A_c}$, or the upper and lower kern distances of the beam section. The $\frac{1}{F_o}$ — intercepts are $+\frac{1}{A_c(f^b_{D+L} - f^b_{tp})}$, $-\frac{1}{A_c f^t_{tp}}$, $-\frac{1}{A_c f_{cp}}$ and $-\frac{1}{A_c(f^t_{D+L} + f_{cp})}$ respectively. The $F_o e$ region can thus be easily constructed by laying out these intercept distances along the axes.

The boundry equations can also be treated as linear by using F_o and $(F_o e)$ as the variables. The curves would then become two pairs of parallel straight lines with slopes of $-\frac{Z_{bb}}{A_c}$ and $+\frac{Z_{bb}}{A_c}$ respectively.

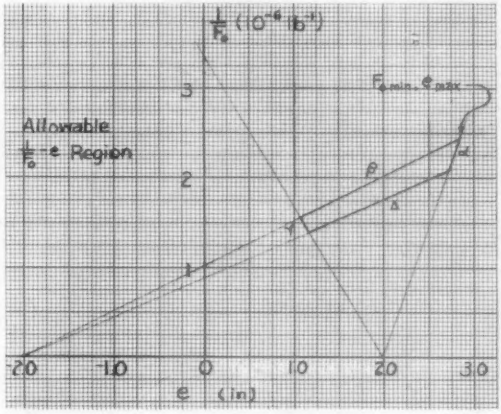
The allowable F_o-e region for the 30 ft. beam, constructed in the above mentioned manners, are as shown in the accompanying chart.

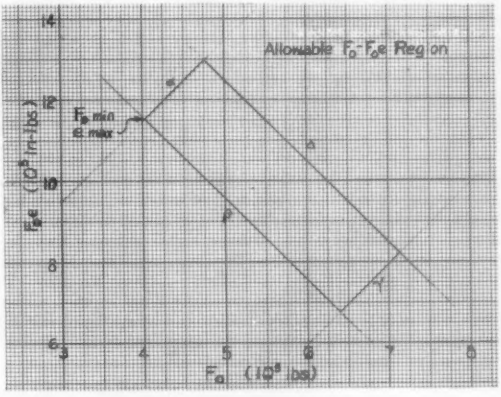
Examining the four boundary equations closely, it can be seen that the intersection of curves α and β corresponds to minimum F_o. It can also be seen that curve β represents the lower limit for F_o (or upper limit for $\frac{1}{F_o}$) any given value of e. Therefore, for prestress force modification, it is only necessary to solve for F_o from equation (15′) or (β).

For the authors' illustrative example, $e_R = 2.77$ inch, $Z_{bb} = 1146$ in^3, $f^b_{D+L} = 1713$ psi, $f^b_{tp} = 0$, and $A_c = 570$ in^2. Substituting into (β)

$$2.77 = 1146\,(1713)\left(\frac{1}{F_o}\right) - \frac{1146}{570}$$

$$F_o = \frac{1146\ (1713)}{2.77 + \frac{1146}{570}} = 410{,}600 \text{ lbs.}$$





Authors' Closure

by **K. K. Kienow** and **C. A. Byran**

Professor Huang's construction is correct, and, as he pointed out, the reciprocal plot was first developed by Magnel.

The authors feel, however, that the direct F_o-e plot we have devel-

oped presents an immediate visual indication of the effect [in terms of the mode of failure] of variations in prestress-eccentricity combinations which is obscured or lost in the reciprocal function or rectangular hyperbola plots. For example, it is immediately obvious from the direct F_o-e plot that having determined $F_{o\,min}$ and e_{max}, any increase in F_o must be accompanied by an associated decrease in eccentricity or the allowable top fibre tensile stress in the end of the beam will be exceeded. This same conclusion may also be reached from the reciprocal or rectangular hyperbola plots, since all three are mathematically identical, but it is not quite as apparent.

In determining the minimum prestress force required for a given eccentricity less than e_{max}, the use of equations [31] and [32] provides a check on the condition of the top fibre at mid-span under full load, which may be overstressed in compression at eccentricities less than e_{max}. This problem is encountered often if the magnitude of the load is approaching the maximum for the section. In this case the eccentricity at the intersection of curves β and γ may be only a few tenths of an inch from e_{max}. The use of the β equation is suggested by Professor Huang will, therefore, not always result in a valid prestress force.

Suppose that in the prestress force modification example we had chosen a revised eccentricity e of 0.5 inches. The β equation will give a corresponding value of F_o of 782,100 lbs. This point lies on the curve β, but beyond the point of intersection with γ, and the top fibre midspan compression would be 2473 psi, overstressed in compression by about 500 psi. Inspection of the F_o-e region plot will show that the point F_o = 782,100 lbs., e = 0.5 in., lies outside of the allowable region. However, as was pointed out in our concluding paragraph, the usual case will not require the plotting of the F_o-e region curves, and the overstressed top fibre condition would not be apparent without further calculations. Therefore, equations [31] and [32] should be used with their built-in check on the midspan top fibre stress; or the fibre stress should be checked independently in conjunction with the β equation method suggested by Professor Huang, before any detailed design calculations are performed for the section.

Discussion

Positioning Drape Points in Prestressed Members*

by **Meryle L. Cochrane**

I would like to take this opportunity to compliment the author for the fine job he did in presenting his depression equations and make some comments of my own which I hope will add to the obvious value of his article.

My first remarks concern omissions and errata which tend to make the derivation difficult to follow for one not familiar with the problem.

(A) Page 77; the equation occurring above Equation (2):

*June, 1961, Vol. 6, No. 2 by David Morris.

$K_x = \frac{8M_E X}{2L^2 M_E}(1 - X)$ should read

$K_x = \frac{8M_E X}{2L^2 M_E}(L - X)$

(B) Since obviously Φ is greater than 5°, tan Φ does not equal Φ and therefore the equality at the upper right hand of page 77 cannot be obtained, and it should be written:

$$\frac{d}{dx}\left[\frac{4X}{L^2}(L - X)\right] = \tan \Phi = \text{slope},$$

since the slope is *defined* as the tangent of the angle of inclination. This correction should prevail throughout the equations following the one mentioned above.

(C) Mr. Morris' statement preceeding equation (4), page 77, which asserts that the equation for K_x may be written as $K_x = K_o + \Phi(x)$ (which should read $K_x = K_o + \tan \Phi\ (x)$) is incorrect. For any given set of design conditions tan Φ is obviously a constant and therefore, the function $K_o + \tan \Phi\ (x)$ is linear and cannot possibly represent a second degree curve such as K_x. The equation that Mr. Morris has written is of the standard form:

$$y = mx + b$$

and is in fact the equation of line OXA. However, $K_x = K_o + \tan \Phi\ (x)$ at the point of tangency of OXA and K_x. This equality is the key to the line of reasoning which should have been applied which is as follows:

For the conditions shown in Figure 3, page 76, to occur, we may state the following:

(1) OXA = K_x at the point of tangency and therefore $K_x = K_o + \tan \Phi\ (x)$

(2) Slope of OXA = Slope of K_x

Writing (1) and (2) as equations in the proper form and solving them simultaneously will yield the result listed on page 78 as equation (7) which states that:

$$a = \frac{K_M - K_o}{4(1 - \sqrt{K_o})} L$$

which is precisely correct but not for the reasons stated by Mr. Morris. Note that equation (7) may be written in the form:

$$a = \frac{2(M_m{}^R - M_o{}^R)}{WL - (8WM_o{}^R)^{1/2}}$$

which may or may not be a more convenient form.

Inherent Assumptions

Mr. Morris' statement at the lower right of page 79 which asserts that the practical limits include: $1.0 \leqq K_m \leqq 2.0$ assumes that the maximum moment curve should be enveloped by the prestress resisting moment curve. This is an assumption that limits the scope of his equations. Quite often a member is designed to carry a good portion of the live load as a cracked section to reduce no-load camber. Thus the depression points must move toward the bearing points an amount determined by the design criteria. From my remarks on the reasoning of the derivation, it should be clear that OXA, K_m, and K_x are independent of each other and therefore equation (7) is applicable for finding "a" for a partially prestressed section such as the one I have described.

Ultimate Moment Capacity

The fact that the service load moment curve is enveloped either wholly or partially by the prestress moment curve does not guarantee that the ultimate load moment curve will be suitably cared for. It is possible to design a cable trajectory which will produce a bending failure at some point other than at midspan when the moment at midspan

is less than the design ultimate. Therefore care must be taken to insure that the depression distance at ultimate load is greater than or equal to the depression distance at design load. This can be done by rewriting equation (7) as:

$$a_u = \frac{2(M^R{}_{M_u} - M^R{}_{o_u})}{W_u L - (8W_u M^R{}_{o_u})^{1/2}}$$

which can be converted to the same type of form as equation (7) on page 78. The equation that I have written for a_u is based on the assumption that the ultimate moment is calculated using the rectangular stress block approximation. This renders the ultimate moment capacity a linear function of gross strand eccentricity.

As a matter of fact, the ultimate harping distance was one of the factors considered in a recent load test performed by the design office where I am employed. The results of that test justify my previous statements with regard to ultimate harping.

Authors' Closure

by **David Morris**

I am grateful to Mr. Meryle L. Cochrane for his comments and critcisms concerning the recent article entitled "Positioning Drape Points In Prestressed Members." I would like to review his comments and also point out additional errata which should be noted.

Referring to Mr. Cochrane's lettered discussions: (A) The equation should be included in the eratta, corrected to read as shown. (B) A point well taken! Tangent Φ should replace Φ in the derivations. (C) There is a misunderstanding here. The equality above equation (4) refers only to the ordinate to the forementioned point of tangency and is not intended to describe the second degree curve. Here, as Mr. Cochrane states, the slope of line OXA and the slope of curve K_x are equal. This formulation must actually precede the establishment of the point of tangency. The approach to the problem involves the defining and equating of the common ordinates to point X, one for the line OXA, the other for the parabolic curve. Herein lies the "key." The equation for "a" as rewritten by Mr. Cochrane provides a fourth means of determining the desired answer. Equations (7), (8) and the chart in Figure 5 are the other alternatives.

The paragraph *Inherent Assumptions* requires clarification. As stated in the paragraph under equation (8), page 78, the limits are intended to facilitate the graphical presentation of Figure 5, and are established in order to present a meaningful visual aid. Actual K values falling outside the chart boundries by no means limit the equation itself. Moreover, the resisting moment, and consequently, the K values need not be based on fiber stresses derived by the elastic theory. As Mr. Cochrane brings out, the method is applicable to a cracked section design and, under certain conditions, it can be used to check ultimate capacity. (See *Ultimate Moment Capacity*)

The timely comments under *Ultimate Moment Capacity* call for further reflection even though this topic is outside the scope of the original paper. Mr. Cochrane suggests using the rectangular stress block approximation in order to utilize the concept of the linearly shaped resisting moment diagram. While this approach is in keeping with described method, it may or may not indicate a safety factor greater than that obtained by applying the recommend-

ed P.C.I. Code design criteria for ultimate moment capacity. In this latter case the resisting moment diagram will consist, in part, of a series of third degree curves which follow a concave trajectory. This will reduce the area under the envelope and lessen the ultimate capacity in some areas. While this third degree curve envelope does approach a straight line in cases where the centroid of the prestressing force follows a linear path, it is evident that inconsistencies will exist when different ultimate theories are applied to the method in question. In view of the various factors involved, it seems appropriate, under present design criteria, that the critical points be found by the elastic theory and checked by the ultimate theory and then, if the situation suggests, several other points could be established in a "local" envelope.

The overall value of Mr. Cochrane's discussion certainly adds to and further clarifies the intent of the original paper. I would like to thank him for his contributions and add my endorsement to his comments suggesting further investigation of strand trajectories, particularly in the field of ultimate capacity.

Errata:

1. Second line below
$$Km = M_M{}^R / M_E,$$
page 75, change "proporting to Proportion.
2. Second line from bottom, left column, page 81, change ".458" to .458L.

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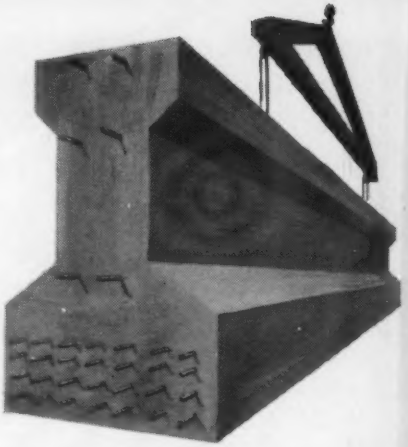

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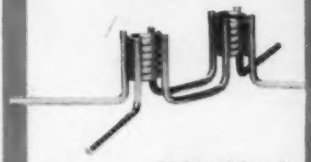

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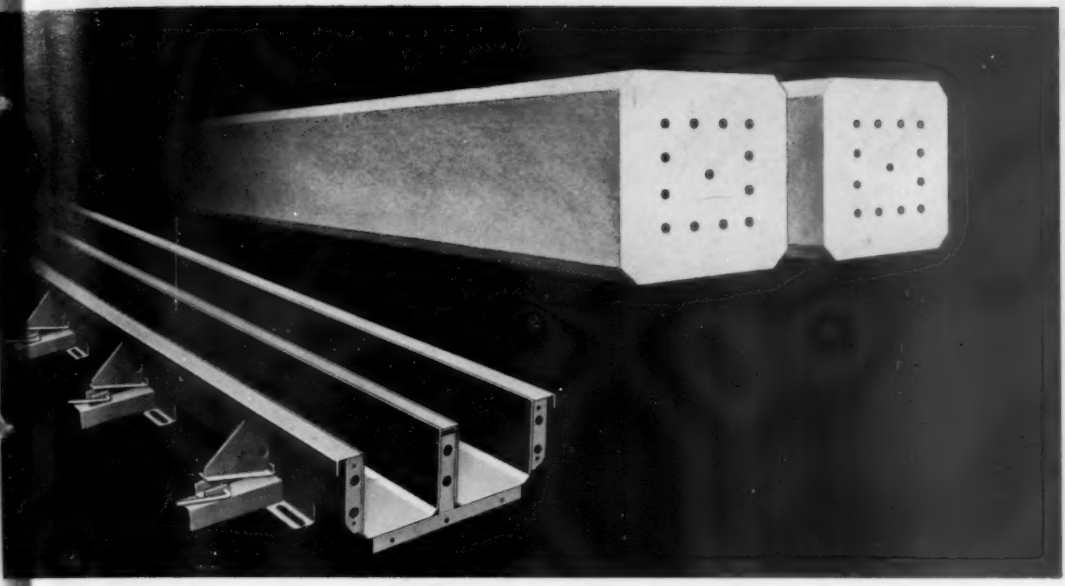

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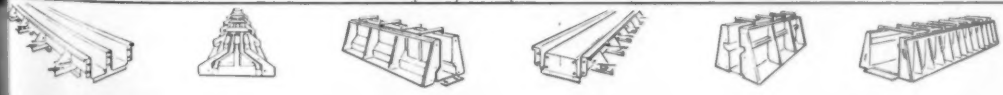

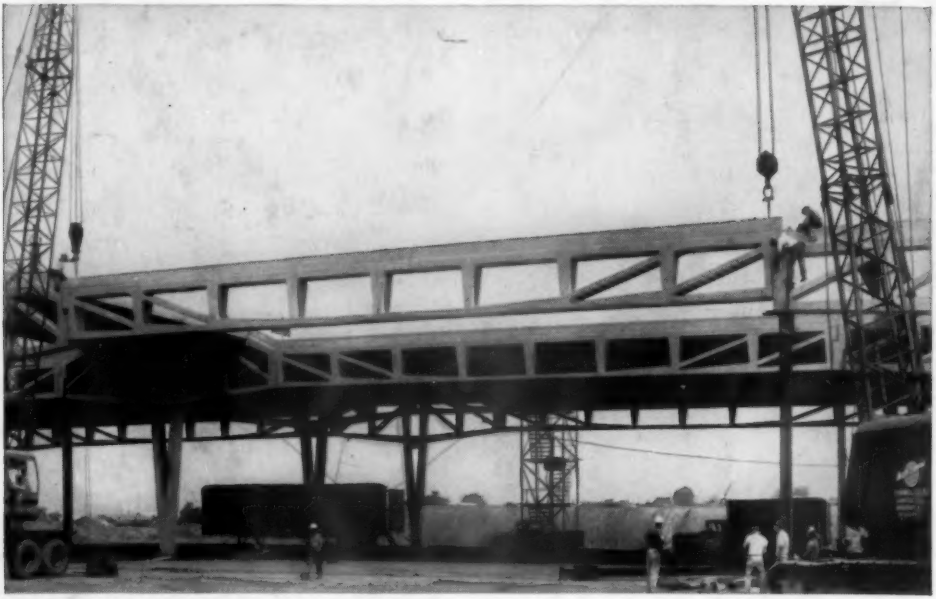

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